SD91-15-F

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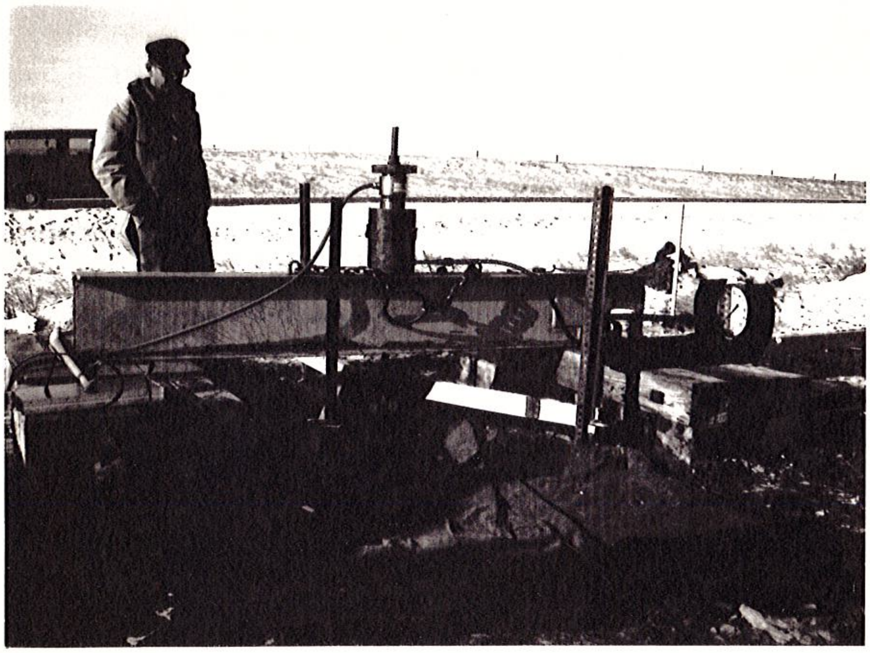

Predicting Skin Friction for Pile Design

Study SD91-15
Final Report

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August, 1992

TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
SD91-15-F		

4. Title and Subtitle	5. Report Date
Predicting Skin Friction for Pile Design	August 31, 1992
	6. Performing Organization Code
7. Author(s)	8. Performing Organization Report No.
Dr. John P. Turner and Mr. Eric W. Sandberg	
9. Performing Organization Name and Address	10. Work Unit No.
Department of Civil and Architectural Engineering University of Wyoming Laramie, Wyoming 82071-3295	11. Contract or Grant No. 310060
12. Sponsoring Agency Name and Address	13. Type of Report and Period Covered
South Dakota Department of Transportation Office of Research 700 East Broadway Avenue Pierre, SD 57501-2586	Final May 1991 to August 1992
	14. Sponsoring Agency Code

15. Supplementary Notes

16. Abstract

This study presents the results of eighteen field uplift load tests on drilled shaft foundations. The tests were conducted to evaluate the reliablity of a procedure used by the SDDOT to evaluate side resistance of deep foundations. This procedure utilizes a field test in which a drill rod is driven into the ground and then load tested in uplift. The uplift load divided by the cylindrical area of the rod is taken as the maximum available unit side resistance. Five test sites were underlain by cohesive soils and one site by cohesionless soil. Three test shafts were constructed and instrumented at each site. Procedures, materials, and equipment used for construction and load testing of the shafts are described.

Side resistances measured by the load tests are compared to those predicted by the SDDOT field pull test. The results exhibit considerable scatter, and this is attributed in part to construction difficulties that resulted in voids in nine of the eighteen shafts. If the defective shafts are not included, the relationship between predicted and measured unit side resistance shows moderately good agreement, based on regression analysis. It is recommended that SDDOT continue to use the field pull test, however, additional load tests, including tests on full-scale drilled shafts, are recommended to further evaluate the reliablity of the pull test. It is recommended that other methods for evaluating side resistance, based upon soil shear strength properties and in-situ tests, be explored.

17. Keywords	18. Distribution Statment
foundations, drilled shafts, piling, skin friction	No restrictions. This document is available to the public from the sponsoring agency.

19. Security Classification (of this report)	Security Classification (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified		

ACKNOWLEDGMENTS

The authors greatly appreciate the support of SDDOT personnel during the completion of this study. D. Ormesher, SDDOT Research Engineer, was the Project Engineer and provided assistance with all aspects of the research. K. Griese and V. Bump, SDDOT Foundation Engineers, supervised all site characterizations and construction of test shafts, and assisted with load testing. Other personnel who assisted with various aspects of field operations included J. Hammell, S. Hinker, C. Lesner, R. Longbons, B. Olson, L. Sowards, and W. Suzle.

South Dakota Department of Transportation and the authors wish to thank Jesse Barrido and Dywidag Systems International, Lemont, Illinois. Their suggestions, support, and donations of Dywidag threadbar is appreciated.

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necesssarily reflect the official views or policies of the South Dakota Department of Transportation, the State Transportation Commission, or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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Chapter I

INTRODUCTION

Deep foundations, including drilled shafts and driven piles, are used extensively in highway construction by the South Dakota Department of Transportation (SDDOT). The principal considerations in deep foundation design are economics and reliability. The objective is to design and construct the most cost-effective and reliable foundation possible.

Side resistance, or skin friction, is one of the critical parameters needed to design a deep foundation. Side resistance is mobilized when a deep foundation is loaded in compression or uplift, and is analyzed as a problem in determining the shearing resistance of the foundation-soil interface acting over the surface of the foundation. According to SDDOT, the method currently in use for predicting side resistance, based on a field pullout test, may not yield representative values of side resistance. This uncertainty is based on load tests conducted on 5 ft long sections of drilled shafts in Gregory County, SD, in 1986. Unit side resistances measured in the load tests ranged from 1,358 to 1,867 psf, while side resistances predicted by the field pull test procedure ranged from 2,400 to 4,300 psf (SDDOT 1986). Uncertainty in design values of side resistance diminishes the reliability and cost-effectiveness of deep foundations. This study was undertaken to evaluate the reliability of the SDDOT procedure for determining design values of deep foundation side resistance.

and varied in consistency from stiff soil to soft rock. At each site, SDDOT conducted a standard site investigation, including the pull test. Three test shafts were then constructed by SDDOT at each test site, and each shaft was load tested to failure in uplift, in accordance with ASTM D-3689 (ASTM 1991). The test results were analyzed to assess the reliability of the pull test for predicting side resistance.

Laboratory interface direct shear tests were conducted on soil samples obtained from each site. Interface conditions were modeled by casting cement mortar against the specimens. Based on analysis of the field and laboratory test results, recommendations are made that may result in improved design methods for deep foundations by the SDDOT.

SCOPE OF REPORT

A review of the literature and evaluation of SDDOT field testing methods for the determination of side resistance are provided in Chapter 2. Chapter 3 presents a description of the test sites, shaft construction, load testing procedures, and laboratory tests. Results are presented and analyzed in Chapter 4, and Chapter 5 presents the summary, conclusions, and recommendations.

Chapter II

REVIEW OF THE LITERATURE AND EVALUATION OF THE SDDOT PULL TEST

This chapter is a review of the available literature on deep foundations side resistance under axial loading, followed by a description and critical evaluation of the SDDOT field pull test.

BEHAVIOR OF DEEP FOUNDATIONS UNDER AXIAL LOADING

When a deep foundation is loaded in compression, the applied loads are resisted by a combination of tip resistance and side resistance. Various methods have been presented in the literature to analyze these two components of resistance. In recent years, significant progress has been made through research and load testing. Realistic analytical and design methods based upon sound behavioral models are available for many loading situations. The following sections review the fundamental mechanisms of load transfer in deep foundations and available methods for analyzing side resistance. Much of this material is a summary of the work by Kulhawy et al. (1983) and Kulhawy (1991) and incorporates information from design manuals on drilled shafts and piles by Vesic (1977) and Reese and O'Neill (1988), as well as recent findings by other researchers.

MECHANISMS OF AXIAL LOAD TRANSFER

A compressive load, Q_c, applied to a vertical deep foundation is resisted by a

The tip and side resistances develop as a function of foundation displacement and the maximum values of each generally occur at significantly different displacement. When the foundation moves downward, side resistance is mobilized. Figure 2-1b illustrates the load transfer and Figure 2-1c shows the corresponding unit side resistance, given by:

$$f(z) = -\frac{1}{P}\frac{dQ_s}{dz} \tag{2-2}$$

in which f = unit side resistance, P = foundation perimeter, and z = depth. At small displacements, most of the load is supported by side resistance. With increasing load, larger displacements occur and the full side resistance is reached. From this point on, further applied loads are resisted by the tip. Figure 2-1b illustrates this phenomenon schematically, from the dotted line at small loads to the solid line at large loads. If no strain-softening occurs in the soil, the unit side resistance (Figure 2-1c) will remain constant.

Figure 2-2 illustrates a typical load versus displacement curve for a deep foundation under compression. It can be seen that the full side resistance is mobilized at relatively small displacements, while relatively large displacements are needed to mobilize tip resistance. The displacement needed to mobilize the maximum side resistance is in the range of 0.4 to 0.6 in. (10 to 15 mm), and is relatively independent of the foundation or soil type, or the foundation dimensions. The displacement necessary to mobilize the maximum represents a summation with depth of $\tau(z)$ times the foundation surface area. The value of τ depends upon on the mode of loading and soil type, as described below.

<u>Drained Loading</u>.

For drained conditions, which develop under most loadings in coarse-grained soils such as sands and for long-term sustained loading of fine-grained soils, soil cohesion $c = 0$ and Equation 2-3 is expressed as:

$$Q_s = \pi\ B \int \overline{\sigma}_h(z) \tan\delta\ dz \tag{2-4}$$

in which: B = shaft diameter, $\overline{\sigma}_h$ = horizontal effective stress, which acts as a normal stress on the soil-foundation interface, and δ = effective stress angle of friction for the soil-foundation interface. Expressing the horizontal stress in terms of the overburden stress ($\overline{\sigma}_v = \overline{\gamma} z$) we have:

$$Q_s = \pi\ B \int \overline{\gamma}\ z\ K(z) \tan\delta\ dz \tag{2-5}$$

in which: $\overline{\gamma}$ = effective unit weight of soil (submerged below the water table; dry or moist above) and K = coefficient of horizontal soil stress ($= \overline{\sigma}_h/\overline{\sigma}_v$). The terms in Equation 5 are often grouped to yield two common alternative forms:

$$Q_s = \pi\ B \int \overline{\gamma} z\ \beta(z)\ dz \tag{2-6}$$

TABLE 2-1. Interface Friction Angles (Source: Kulhawy et al. 1983)

Interface Materials	Ratio of Interface Angle of Friction to Soil Angle of Friction, $\delta/\overline{\phi}$
Sand/rough concrete	1.0
Sand/smooth concrete	0.8 - 1.0
Sand/rough steel	0.7 - 0.9
Sand/smooth steel	0.5 - 0.7
Sand/timber	0.8 - 0.9

horizontal stresses, and the stress changes caused in response to construction, loading, and time. Taking these factors into account, there may be either an increase or decrease from the original in-situ K_0, the coefficient of horizontal soil stress at rest. Values of K reported in the literature range from a low of about 0.1 to a high of around 5.0. These bounds correspond roughly to the range from minimum active to maximum passive stress states. Similarly, a wide variation in the values of β and f are reported in the literature.

Ultimately the side resistance of a deep foundation is controlled by the in-situ stresses. At the present time there are three approaches to evaluate K_0. The first is field measurement using in-situ devices such as the pressuremeter or dilatometer, which can be used to provide an estimate of the in-situ K_0. The second approach is to estimate K_0 based on a knowledge of the stress history of the soil and empirical correlations of soil behavior. One widely used correlation presented by Mayne and Kulhawy (1983) is given by the following:

displacement piles (H-piles, open end pipe) will not allow as much relaxation as a drilled shaft. Driven large displacement piles (closed end pipe) will increase the horizontal stresses. No detailed study of K/K_o has been made for driven piles, but the recommendations given by Kulhawy et al. (1983) and presented in Table 2-2 are in qualitative agreement with the limited data available.

TABLE 2-2. Horizontal Soil Stress Coefficients
(Source: Kulhawy et al. 1983)

Foundation and Method of Installation	Ratio of Horizontal Soil Stress Coefficient to In-Situ Value, K/K_0
Jetted Pile	1/2 to 2/3
Drilled shaft, cast-in-place	2/3 to 1
Driven pile, small displacement	3/4 to 1 1/4
Driven pile, large displacement	1 to 2

Beta (β) Factor. As an alternative to evaluating δ and K, side resistance is sometimes estimated as a function of β. In this approach, the β factor lumps together the coefficient of horizontal stress and the interface angle of friction. Reese and O'Neill (1988) recommend the following empirical expression for β, based on the results of load tests on slender drilled shafts in sand:

$$\beta = 1.5 - 0.135\ z^{0.5}, \quad 1.2 \geq \beta \geq 0.25 \qquad (2\text{-}10)$$

field and that all phenomena which occur during construction and loading can be lumped together into one "constant" (Kulhawy 1991). α often is used in practice because of simplicity and because the real problem is complex and not yet understood completely. Values of α vary with foundation type and installation technique.

Several correlations to obtain α for driven piles have been published in the literature. One of the best known is that of Tomlinson (1980), given as Figure 2-3. For drilled shafts, Kulhawy and Jackson (1989) obtained the correlation shown in Figure 2-4, based on analysis of 106 field load tests on drilled shafts in cohesive soils, under both compression and uplift loading. In this figure, s_u is normalized by p_a = atmospheric stress in the desired stress units (2,116 psf, 101.3 kN/m^2, etc.). Tomlinson's original 1957 proposed α correlation curve also is included in Figure 2-4. As can be seen, the range in capacity, geometry, and soil strength is broad, and the data for uplift and compression are roughly comparable.

An alternative recommendation for α is given by Reese and O'Neill (1988), for drilled shafts in cohesive soils. A constant value of $\alpha = 0.55$ is recommended, except for the top 5 ft and bottom one diameter, where $\alpha = 0$. In other words, side resistance along the top 5 ft and along the bottom one diameter of the shaft are neglected. This is based upon uncertainties in load transfer near the ground surface because of seasonal moisture changes, molding away of clay near the groundline under cyclic lateral loading, and other factors. The development of tensile stresses directly above the base of drilled shafts in compression leads to the recommendation for neglecting side resistance for one diameter above the base.

It is important to note that the Reese and O'Neill recommendation is based upon analysis of load tests on instrumented drilled shafts. The use of these recommendations in practice should be limited to similar soil and foundation conditions. No experimental data are available for shafts in clays with s_u greater than 6 tsf, and the results should not be applied to sites with sensitive clays, clay-like shales or mudstones, or expansive clays (Reese and O'Neill 1988). Therefore, the Reese and O'Neill recommendation should not be used for many of the shale formations found throughout South Dakota (e.g. the Pierre shale). In addition, values of s_u of the soils used to derive the recommended α values were determined mostly from unconsolidated undrained triaxial tests on samples of 2 to 3 inches in diameter under an isotropic cell pressure equal to the in-situ overburden stress. If this recommendation is used in design, the method used to determine s_u should produce values equivalent to or less than those obtained from UU triaxial compression tests.

Undrained Shear Strength. In an undrained analysis using total stress methods, the undrained shear strength, s_u, is the most important soil parameter to obtain. s_u is a highly variable parameter which is a function of the effective stress friction angle, stress history of the soil, effective overburden stress, water content, and other factors. Because of this, s_u exhibits a relatively large coefficient of variation (standard deviation/mean), ranging from 29% to 85% (Harr 1977). Due to this relatively high variability, undrained analyses using numerous papers on the practical use of in-situ tests for measurement of soil properties and foundation design.

EVALUATION OF SDDOT FIELD TEST

This section presents an evaluation of the current SDDOT method for determining design values of side resistance for deep foundations. A brief description of the test is followed by an interpretation of the test results and its applicability to various deep foundation types, soil types, and loading conditions.

Test Description

The field test used to evaluate side resistance is conducted as part of the SDDOT standard subsurface investigation procedure, as described by Bump et al. (1971). First, a preliminary investigation is conducted by augering a hole using a 4-1/2 inch continuous flight auger to determine major soil zones and the location of the water table. The auger borings normally extend 20 or more feet below the anticipated tip elevation of the foundation. This information is used by the field engineer to determine the depths at which samples are to be taken.

Next, a drive test is conducted and samples are obtained using a Failing Drilling Unit. A $2^7/_8$ inch diameter drill rod (P.K. rod) equipped with a California retractable plug sampler is driven by a 490-lb hammer with a drop height of 30 inches (nominal 1225 ft-lbs energy). The number of blows per foot required to drive the rod is observed and recorded.

$$SFV = R / .7527*L \tag{2-13}$$

in which: SFV = skin friction value (pounds per square foot), R = load cell reading (pounds), L = depth of the sampler tip below the surface (feet), and π x diameter of the drill rod = .7527 ft (SDDOT 1964).

Interpretation of Results

For practical purposes, the drill stem can be considered to be a small-diameter driven steel pipe pile. According to Equation 2-3, the average unit side resistance over the length of the drill stem can be computed by:

$$f = \frac{Q_s - W}{\text{surface area}} \tag{2-14}$$

in which f = unit side resistance, Q_s = total side resistance, and W = weight of the drill stem. If the load cell reading, R, is in pounds, then taking into account the diameter of the drill rod yields the following expression for average unit side resistance, in units of psf:

$$f = \frac{R - W}{.7527\ L} \tag{2-15}$$

in which: L = length of drill stem below the surface, in feet. Equation 2-15 is the same expression given for skin friction using the SDDOT test procedure (Equation 2-13), except recommendation for modification factors by which the unit side resistance should be multiplied to account for interface material for deep foundations in cohesionless soils. It is assumed that the average $\delta/\overline{\phi}$ of the drill stem ranges from 0.7 to 0.8. The factors in Table 2-3 should be modified and updated as additional field load test results are obtained.

For foundations in cohesive soils under undrained loading conditions, corresponding information on the effects of interface material is not available. Referring to Figure 2-3, there is a small difference between the curves for α for concrete piles compared to all piles, but there is a large amount of scatter and the data are insufficient to make any distinctions on the basis of pile type. Until better correlations are developed, values of side resistance obtained from the pullout test should be used directly for deep foundations made of steel, but should be scaled by the ratio of α from the curve for drilled shafts to α from the curve for all piles when designing concrete deep foundations. The appropriate ratio is a function of soil undrained shear strength and therefore requires an estimate of s_u.

TABLE 2-3. Modification Factors for Interface Material

Interface Material	Ratio of Foundation Side Resistance to Side Resistance from Pull Test
Sand/rough concrete	1.25 - 1.4
Sand/smooth concrete	1.0 - 1.4
Sand/rough steel	0.9 - 1.3
Sand/smooth steel	0.6 - 1.0
Sand/timber	1.0 - 1.3

TABLE 2-4. Modification Factors for Method of Installation

Foundation and Method of Installation	Ratio of Foundation Side Resistance to Side Resistance from Pull Test
Jetted Pile	0.4 - 0.5
Drilled shaft, cast-in-place	0.5 - 0.8
Driven pile, small displacement	0.6 - 1.0
Driven pile, large displacement	0.8 - 1.6

soils and during long-term, sustained loading of fine-grained soils. Undrained conditions develop during short-term loading of fine-grained soils. The SDDOT pull test consists of short-term loading. Therefore the soil response will be drained for cohesionless soils (sands) and undrained for cohesive soils (clay).

For sites consisting predominantly of sand, the results are best interpreted within the context of the β method (Equations 2-4 to 2-8). In this formulation, the unit side resistance is given by:

$$f = \beta\, \overline{\gamma} z \tag{2-16}$$

where $\beta = K \tan\delta$. Accordingly, the unit side resistance obtained from the pull test should be modified by the factors given in Tables 2-3 and 2-4 to account for changes in β caused by differences in the interface material and construction effects.

Sources of Variability

Many sources of variability exist which can affect the results of field tests. These include variability in the site conditions and soil properties, as well as sources of error and variability inherent in the test procedure.

Variability in Site Conditions. Subsurface ground conditions can range from regular to highly variable. Regular would describe sites with fairly uniform soil deposits, including layered sites where the thickness of layers does not vary significantly across the site and variability in the horizontal direction is not significant. Variable conditions can be described by two limiting conditions: consistent or erratic. Consistent variations include sites with layers of variable thickness where the variations are regular and can be predicted using data from several boreholes. Erratic variations occur when highly variable conditions exist in both the horizontal and vertical directions and the variations are difficult or impossible to predict. An example would be unsorted glacial till consisting of a wide range of particle sizes (clay to boulders) and a high degree of horizontal variability.

The degree of subsurface variability directly affects the use of the pull test results. For regular sites consisting of basically one soil type, the test results should be applicable to the design of deep foundations across the site, provided all foundations are constructed to approximately the same depth as the pull test and in the same soil. For sites that are layered with little horizontal variability, the test results also should be applicable to the entire site if the layers are either all sand or all clay soils. For sites consisting of a mixed purposes. Even under carefully controlled laboratory conditions, soil index properties like grain size distribution and Atterburg limits, and performance properties such as shear strength and compressibility, exhibit variability. This variability becomes more pronounced in the field under natural geological conditions. The degree of variability of in-situ soil properties will contribute directly to the variability of the pull test results.

Test Variability. Every field and laboratory test has its own inherent variability. Variability in the pull test can occur as a result of differences in the pulling process. During pulling, variability may be introduced by varying degrees of cable friction, the time elapsed between driving and pulling, and measurement errors due to improper calibration of the load cell.

Variability in the site conditions, soil properties, and testing method combine to produce the overall variability of the pull test results. To quantify the reliability requires statistical analysis of the results of multiple pull tests and load tests on prototype deep foundations. An attempt will be made to conduct such an analysis as part of the research in progress. For a preliminary glimpse of test variability, Table 2-5 tabulates the results of pull tests conducted for this study, including calculation of average unit side resistances according to Equation 2-14. Examination of the average side resistances shows that variability ranges from moderate (e.g., site D) to relatively high (e.g., sites B and F), for the limited number of cases considered.

Drained Analysis (Site B). The total side resistance, Q_s, of a 2 ft diameter, 20 ft long drilled shaft is calculated. The following assumptions are made:

° The friction angle ratio for the drill rod/soil interface is $\delta/\phi = 0.75$

° The friction angle ratio for the shaft/soil interface is $\delta/\phi = 1.0$.

° The rod is a large-displacement driven pile and $K/K_O = 1.25$

° For the drilled shaft $K/K_O = 1$ (high quality construction)

Side Resistance Modification factors:

for interface: $1 / 0.75 = \underline{1.33}$

for installation method: $1 / 1.25 = \underline{0.8}$

Corrected Side Resistances:

B2: 253 psf × 1.33 × 0.8 = 269 psf

B3: 136 psf × 1.33 × 0.8 = 145 psf

B3: 628 psf × 1.33 × 0.8 = 668 psf

Total Side Resistance:

Q_s = Surface Area × corrected unit side resistance

B2: $Q_s = \pi \times 2$ ft × 20 ft × 269 psf = 33,804 lbs ≈ 16.9 tons

B3: $Q_s = \pi \times 2$ ft × 20 ft × 145 psf = 18,221 lbs ≈ 9.1 tons

B4: $Q_s = \pi \times 2$ ft × 20 ft × 668 psf = 83,943 lbs ≈ 42.0 tons

Comments: The range in computed values of side resistance is large (9.1 to 42.0 tons), reflecting the large variability in the test results at this site. Also, the assumed values for the friction angle ratio and horizontal stress ratio are preliminary estimates, not yet validated by field load test results.

Chapter III

SITE CHARACTERIZATION AND TEST PROCEDURES

This chapter describes the test sites, including subsurface profiles, soil test results, construction and instrumentation of test shafts, and load testing procedures. The apparatus and procedures used to conduct interface direct shear tests are also described.

GEOLOGICAL SETTING

Load tests were conducted on drilled shaft foundations at sites located in or near the cities of Rapid City and Pierre, South Dakota. Figure 3-1 shows the general geology of South Dakota. West of the Missouri River and extending to the Black Hills uplift, the principal bedrock units are of upper Cretaceous age, including large areas underlain by the Pierre Formation, a gray to black massive shale that locally contains high amounts of the clay mineral montmorillonite. The city of Pierre is located on the Missouri River, which forms a boundary with the eastern physiographic region of South Dakota, which is characterized by Pleistocene glacial deposits. The load test site near Pierre (site F) is underlain by the Pierre shale.

The remaining five load test sites are located in the vicinity of Rapid City, on the eastern flank of the Black Hills uplift. Figure 3-2 is a general geological map of the Black Hills area. The sedimentary formations on the eastern flank of the uplift dip eastward and therefore decrease in age from west to east. Formations in which load tests were conducted

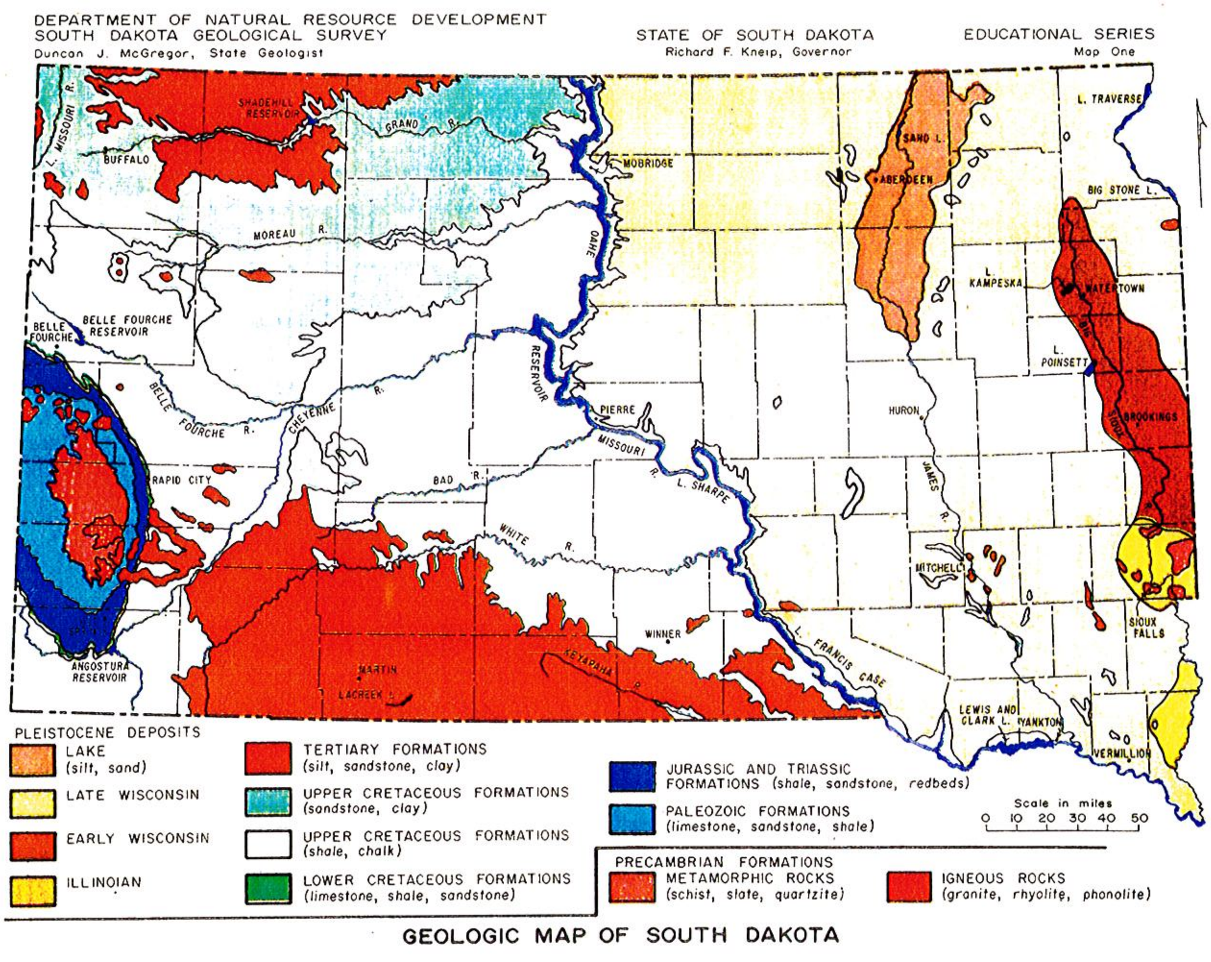


Figure 3-1. General Geology of South Dakota (Source: S.D Geol. Survey).

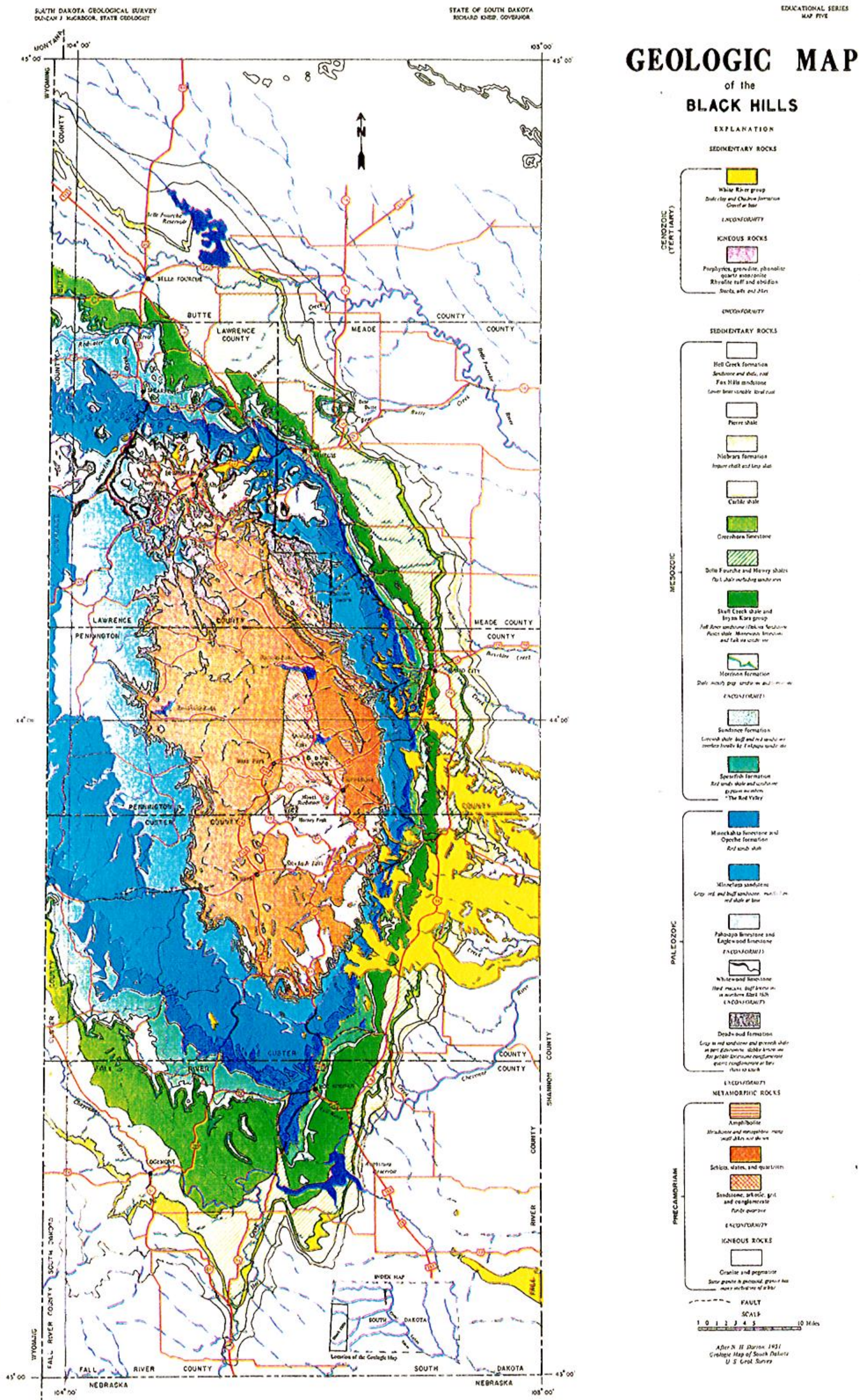


Figure 3-2. General Geology of the Black Hills Area (Source: S.D. Geol. Survey).

are summarized in Table 3-1. The Rapid City test site locations are shown in Figure 3-3, except for Site C, which is located west of Rapid City along I-90.

SITE DESCRIPTIONS

At each site, an auger boring was made to provide a subsurface profile and to determine the location of the water table. At least three SDDOT pull tests were conducted at each site to obtain soil samples and for predicting side resistances. Laboratory testing included grain size distribution and unconfined compressive strength by the SDDOT geotechnical laboratory. Atterberg limits and interface direct shear tests were conducted by the second author at the University of Wyoming. Each site is described briefly below.

TABLE 3-1. General Geological Setting of Test Sites.

Site	Principal Formation or Soil Type	Description	Geological Age
A	Belle Fourche Formation	black shale, weathered	Lower Cretaceous
B	Alluvial deposits in flood plain of Rapid Creek	sand, silt, and gravel	Quaternary
C	Spearfish Formation	red gypsiferous sandy shale and sandstone	Triassic
D	Carlile Formation	gray siltstone and shale	Upper Cretaceous
E	Pierre Formation	light to dark gray shale, weathered	Upper Cretaceous
F	Pierre Formation	light to dark gray shale, weathered	Upper Cretaceous

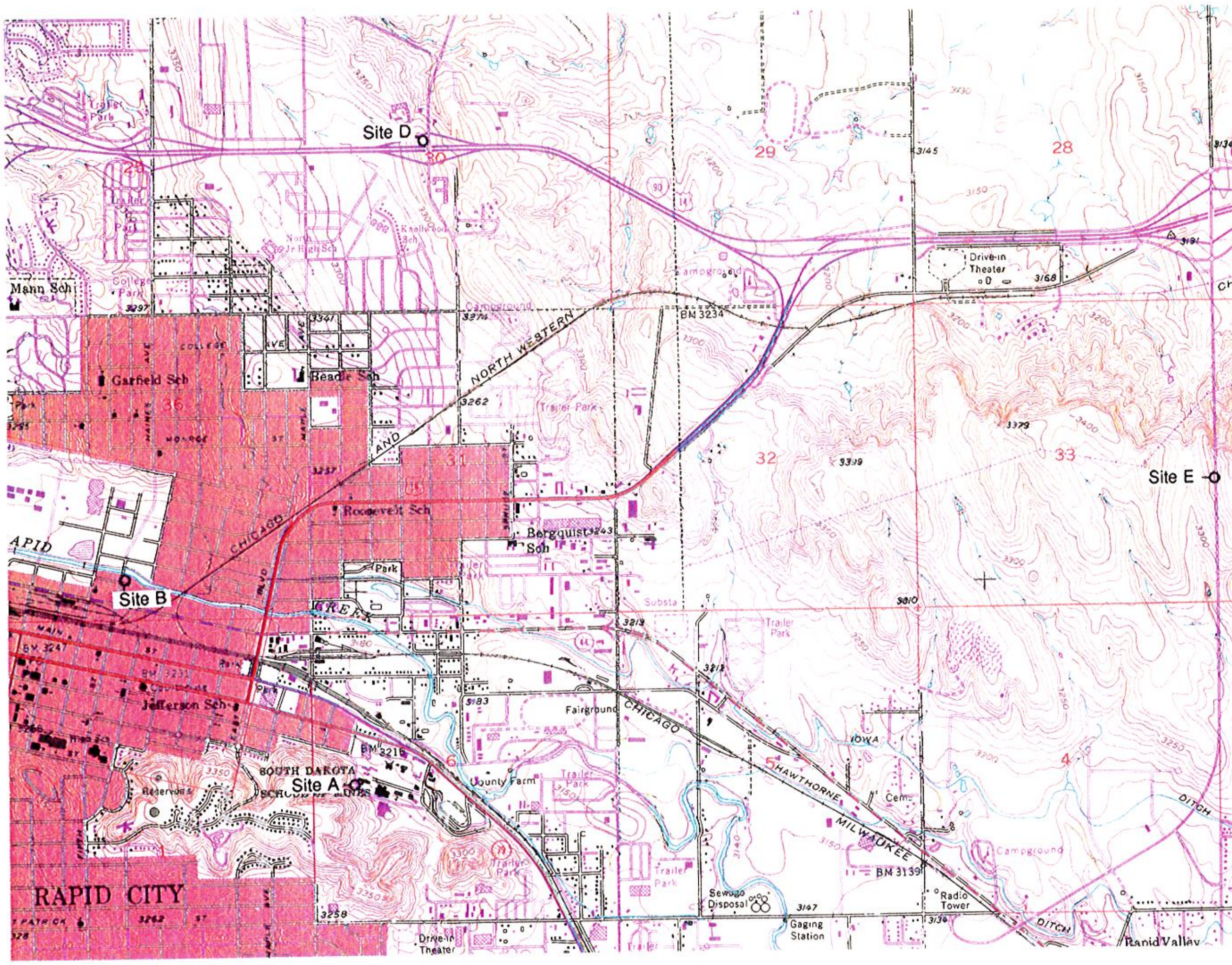


Figure 3-3. Location of Test Sites in Rapid City, SD.

SITE A The subsurface profile at site A, located on the campus of the South Dakota School of Mines and Technology in Rapid City (Figure 3-3), consists of 0.4 ft of gravel underlain by 20.6 ft of black shale of the Belle Fourche Formation, with very wet shale encountered at a depth of approximately 19 ft. Figure 3-4 shows the soil profile (based on a single auger boring) and a plan view of Site A. Three pull tests were conducted at Site A. Difficulty occurred during the sampling phase of Test A2, requiring the drill rod to be pulled to repair the sampler, which was then redriven to 11 ft where the sampler was opened. No problems were encountered during the other two pull tests. Load cell readings and soil property data are summarized in Tables 3-2 and 3-3. Atterberg limits of Sample No. 7 from Test A2 yielded a liquid limit (LL) of 63.3 and plasticity index (PI) of 28.9, resulting in a Unified Soil Classification (USCS) of MH.

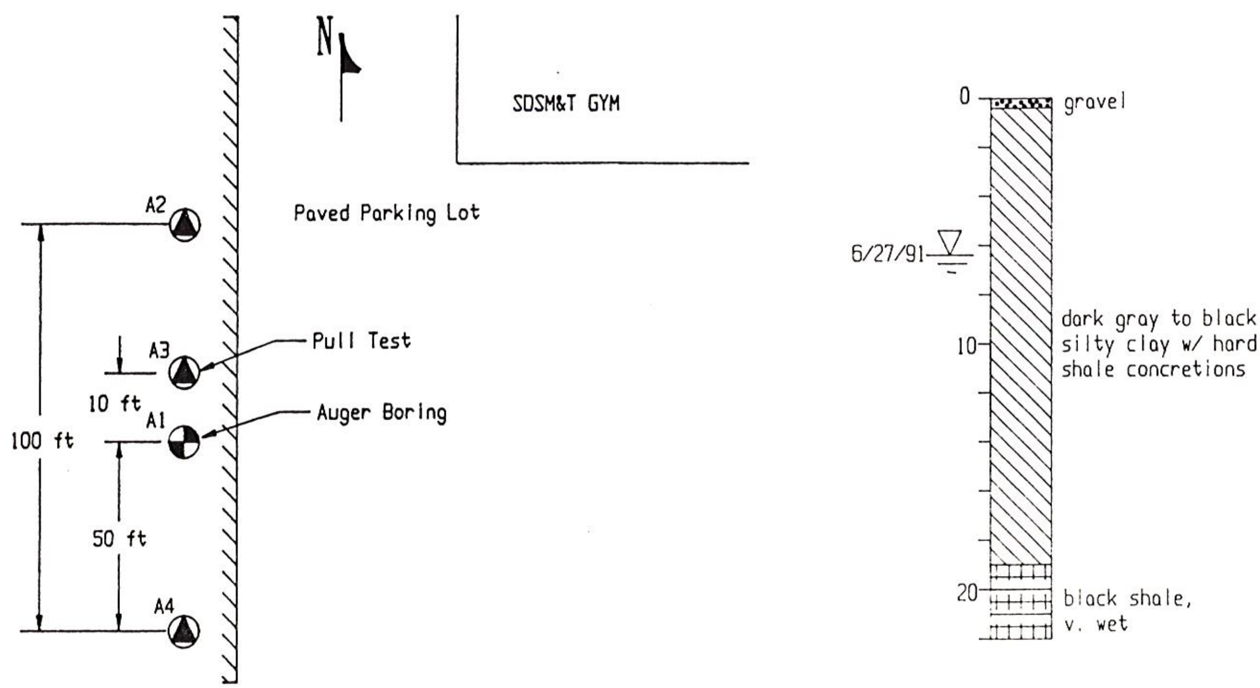


Figure 3-4. Site A.

Three pull tests were conducted at Site B. A cobble was encountered during the driving phase of Test B3, therefore no samples were obtained. However, load cell readings were taken from each pull test. No soil tests were conducted on samples from Site B. Table 3-4 summarizes the pull test results.

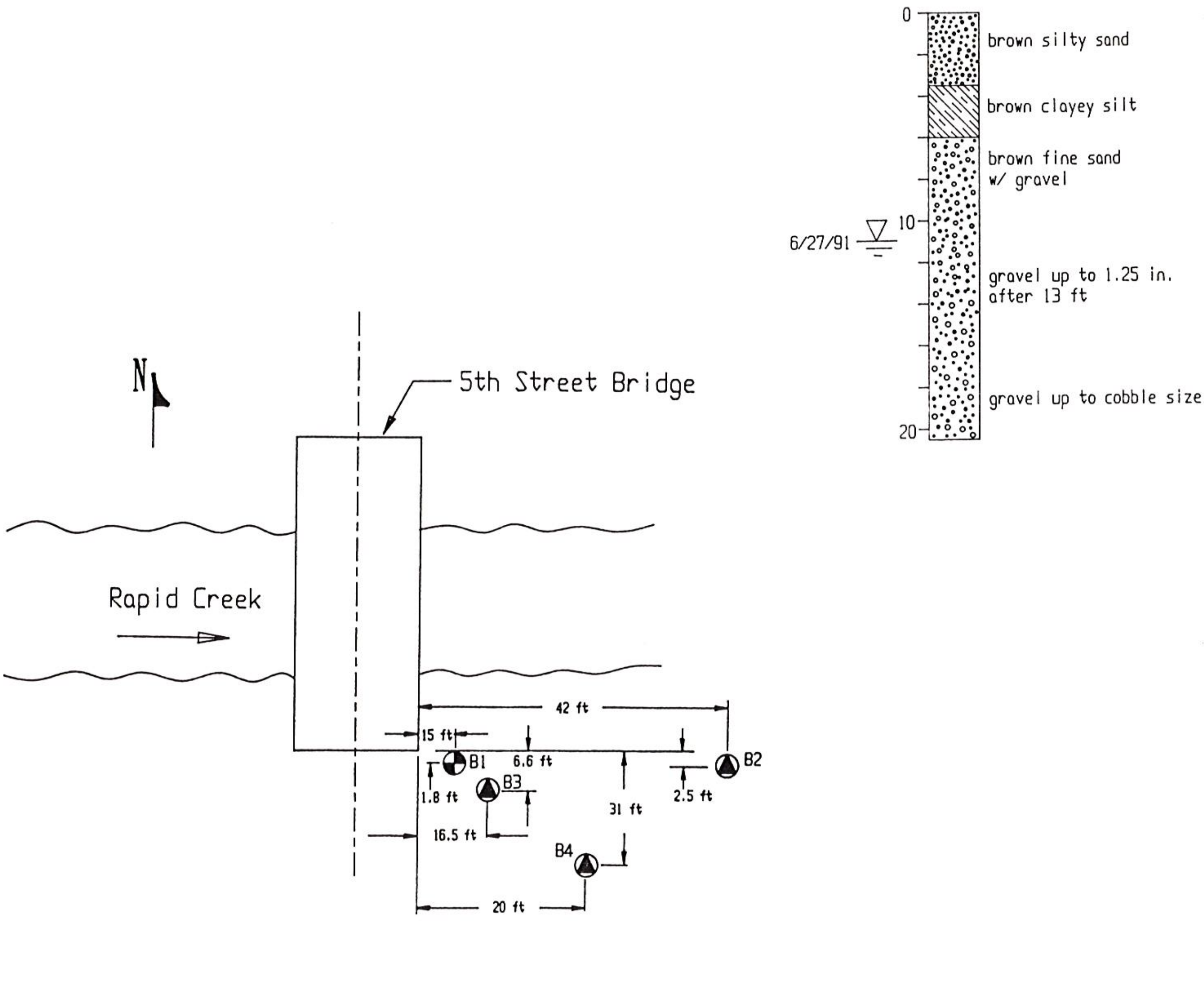


Figure 3-5. Layout and Soil Profile, Site B.

TABLE 3-5. Results of Pull Tests, Site C.

Pull Test No.	Depth (ft)	Dynamometer Reading, R (lb)	No. of Samples	Sample Depths Interval (ft)
C3	8.9	14,900	3	7.9 to 8.9
C4	11.85	18,100	5	10.2 to 11.85

TABLE 3-6. Summary of Soil Properties, Site C.

Pull Test No.	Sample No.	Sample Depth (ft)	s_u[1] (psf)	water content (%)	% Sand	% Silt	% Clay
C4	5	11.0	4,464	8.3	0.6	64.6	34.8

1. s_u = undrained shear strength.

Four pull tests were attempted at Site C, however, only two provided samples and dynamometer readings. Very hard gypsum was encountered during the driving phase of Tests C2 and C2A, and both were abandoned. A summary of the number and depths at which samples were obtained and pull test results are given in Table 3-5. Soil properties are summarized in Table 3-6 and Sample No. 5 from Test C4 was classified as ML, based on its grain size characteristics and Atterberg limits of LL = 27.7 and PI = 10.1.

<u>SITE D</u> Site D is located at the intersection of I-90 and Lacrosse St. in Rapid City (Figure 3-1). The subsurface consists of soft, moist, gray silt and clay (weathered Carlile Formation) to a depth of 17 ft. From 17 to 21 ft a firmer, gray silty clay was encountered. The site layout and soil profiles are shown in Figure 3-7. Pull test results and sample data are summarized in Table 3-7 and soil property data are summarized in Table 3-8. Sample

TABLE 3-7. Results of Pull Tests, Site D.

Pull Test No.	Depth (ft)	Dynamometer Reading, R (lb)	No. of Samples	Sample Depths Interval (ft)
D2	11.0	19,300	6	9.0 to 11.0
D3	10.0	21,400	9	7.0 to 11.0
D4	9.0	17,100	6	7.0 to 9.0

TABLE 3-8. Summary of Soil Properties, Site D.

Pull Test No.	Sample No.	Sample Depth (ft)	s_u[1] (psf)	water content (%)	% Sand	% Silt	% Clay
D2	1	11.0	11,837	13.9	0.4	42.8	56.8
D2	5	9.7	10,958	14.2	0.4	44.8	54.8
D2	4	10.0	-	-	0.6	41.8	57.6
D3	3	9.3	8,669	14.1	1.2	42.0	56.8
D3	8	7.7	7,445	15.3	0.4	38.8	60.8
D4	1	9.0	11,563	13.6	0.4	40.8	58.8

1. s_u = undrained shear strength.

SITE E Site E is located along Saint Patrick Street Bypass in Rapid City (Figure 3-3). The subsurface profile consists of 1 ft of topsoil underlain by firm, light brown silty clay (weathered Pierre Formation) to a depth of 14 ft. Firmer clay with gypsum crystals was encountered from 14 to 20 ft. The soil profile and site layout are shown in Figure 3-8. Three pull tests were conducted at Site E. The sampler would not open during Test E3, so the drill rod was pulled, repaired, and redriven to a depth of 10 ft where the sampler was opened. Pull test results and sample depth intervals are given in Table 3-9, and soil property data are summarized in Table 3-10. Sample No. 6 from Test E3 yielded a classification of CH, based on grain size characteristics and Atterberg limits of LL = 60.6 and PI = 29.5.

TABLE 3-10. Summary of Soil Properties, Site E.

Pull Test No.	Sample No.	Sample Depth (ft)	s_u (psf)	Water Content (%)	% Sand	% Silt	% Clay
E2	1	18.0	3,298	21.4	7.2	24.0	68.8
E2	5	16.7	6,178	21.6	0.2	23.0	76.8
E2	8	15.7	4,162	16.2	20.4	26.8	52.8
E3	1	15.0	8,438	19.1	1.6	25.6	72.8
E3	4	14.0	8,395	16.1	10.2	27.0	62.8
E3	6	13.3	-	-	5.7	24.8	69.5
E4	1	18.0	5,933	22.9	3.0	24.2	72.8
E4	5	16.7	6,379	21.0	2.6	24.6	72.8
E4	6	16.3	5,457	21.6	0.4	22.8	76.8

1. s_u = undrained shear strength.

SITE F Site F is at the junction of SD Highway 1804 and US Highway 14/83 near Pierre, SD. No auger test data are available, however previous investigations by SDDOT at this site indicate that it is underlain by 10 to 12 ft of weathered Pierre shale overlying relatively unweathered Pierre shale. Site layout and the subsurface profile are presented in Figure 3-9, while the pull test results and soil property data are given in Tables 3-11 and 3-12, respectively. Sample No. 5 from Test F1 gave LL = 162.0 and PI = 120.3, yielding a classification of CH.

TABLE 3-11. Results of Pull Tests, Site F.

Pull Test No.	Depth	Dynamometer Reading, R (lb)	No. of Samples	Sample Depths Interval (ft)
F1	16.0	27,700	6	14.0 to 16.0
F2	15.0	14,100	8	12.3 to 15.0
F3	18.0	24,700	10	14.7 to 18.0

DESIGN, CONSTRUCTION, AND INSTRUMENTATION OF TEST SHAFTS

Design of Test Shafts

Test shaft diameters were determined by the size of the drilling equipment available to the SDDOT geotechnical laboratory. Shafts in cohesive soils, which could be constructed without temporary excavation support, were installed using a 6-in. diameter auger. Test shafts at the cohesionless site (Site B) were constructed using a 7-in. diameter hollow-stem auger to allow placement of concrete through the auger as it was removed. Test shaft depths were determined by three factors: (1) all three test shafts at each site were to be the same depth, (2) uplift capacities were to be within the range of SDDOT load testing equipment, and (3) shaft depths should correspond to the depths at which pull tests were conducted. Preliminary estimates of uplift capacity were conducted using the pull test results with the current SDDOT method for predicting side resistance (Equation 2-13 and data given in Table 2-5). Design dimensions based on these criteria are given in Table 3-13.

TABLE 3-13. Design Dimensions of Test Shafts

Site	Design Diameter (in.)	Design Depth (ft)
A	6	14.5
B	7	17.0
C	6	12.0
D	6	12.0
E	6	15.0
F	6	18.0

Construction of Test Shafts

The dry method of construction was used for shafts located at sites with cohesive soils (Sites A, C, D, E, F). A 6-inch diameter hole was excavated to the design depth using a continuous flight auger. Following removal of the auger, a 1-inch diameter threaded Dywidag bar (Grade 150, high strength), with welded centering supports was centered in the hole. Concrete was poured from a ready-mix truck into a timber hopper placed on top of the hole. The hopper directed the fluid concrete vertically down the hole by freefall. Figure 3-10 is a photograph of the concrete placement operation at Site E. The concrete mix design was for a water/cement ratio of 0.4, slump of 6 to 8 in., and 28-day compressive strength of 4,500 psi. Following concrete placement, two pedestals for support of dial gauges were set into the concrete before it hardened.

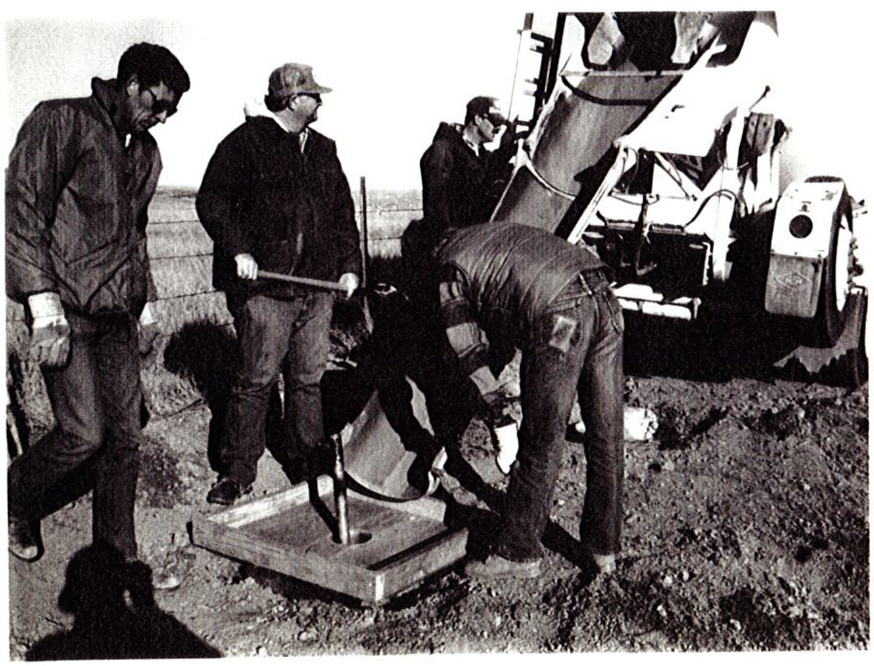


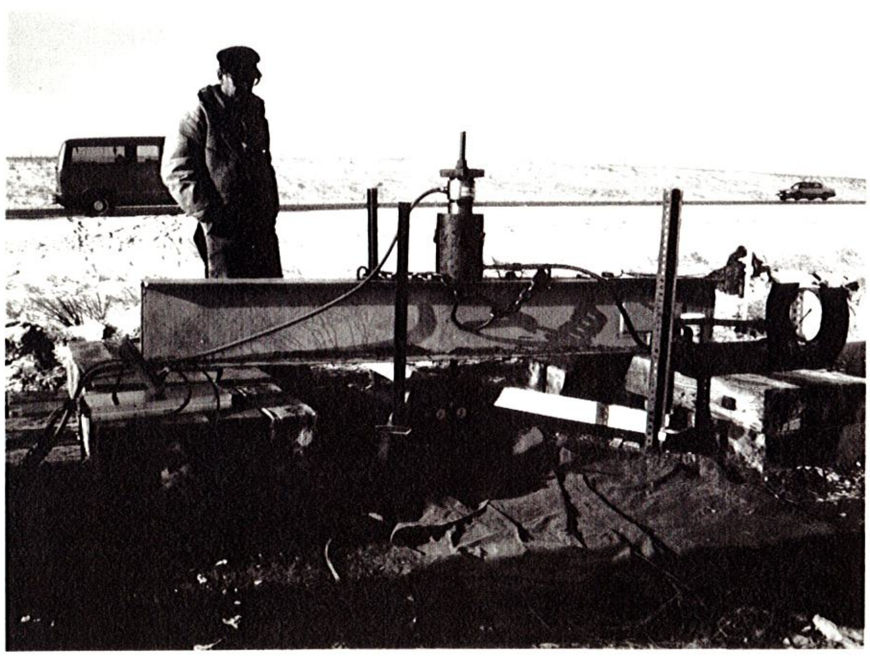
Figure 3-10. Concrete Placement, Site E

At Site B, a 7-inch diameter hollow-stem auger was used for drilling. Upon reaching the design depth, concrete was placed through the hollow stem as the auger was removed.

Construction diffuculties were encountered at several sites and were caused primarily by infiltration of water into the excavations before concrete placement. As observed by the first author, at Site A, water began flowing into each excavation immediately following removal of the drilling auger. Several feet of water may have been present at the bottom of each hole by the time concrete was placed by freefall, which generally results in shafts with defects or voids. At Site D, the excavation at Test Shaft D1 was observed to have approximately 1 ft of water prior to concrete placement. At Site E, all holes were observed to be dry before concrete placement. Neither author was present during construction of tests shafts at the remaining sites, but defects observed in shafts at Sites B and F indicate that construction difficulties occurred. Inspection of the test shafts following load testing is discussed further in Chapter IV.

Instrumentation

One shaft at each site was instrumented to measure axial load transfer versus depth. Instrumentation consisted of strain gages bonded to the single reinforcing bar at depths corresponding to 0.25, 0.50, 0.75, and 1.0 times the shaft length. Strain gages were Model S/ADP-350-300 semiconductor gages with a resistance of 350 ohms and a gage factor of 130. All gage outputs were monitored by a MEGADAC data acquisition system, consisting against the jack by a steel bearing plate and nut. Figure 3-11 is a photograph of the overall load test setup and Figure 3-12 is a closeup showing the arrangement of the jack, load cell, and dial gages.



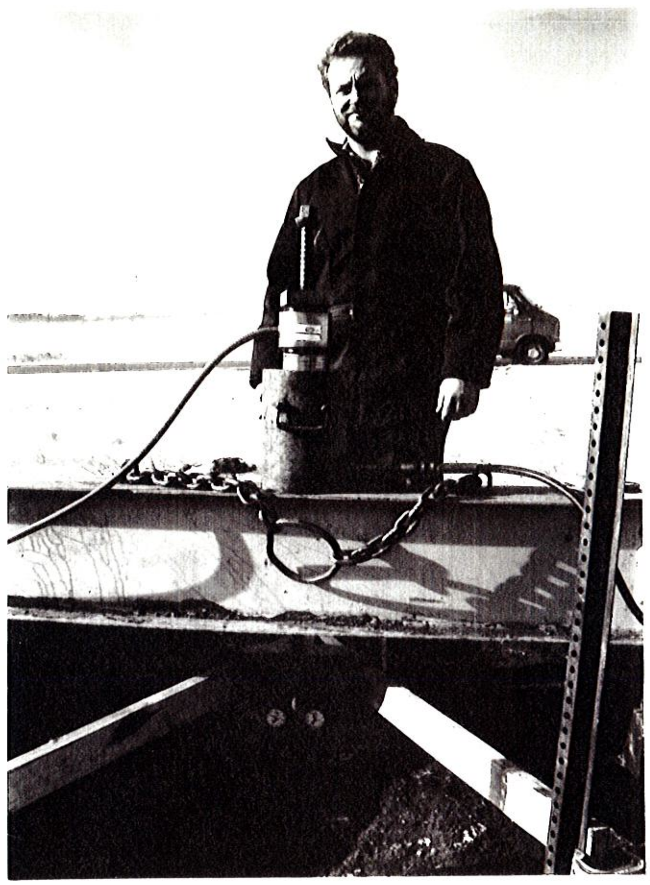


Figure 3-12. Closeup of Load Test Equipment

Loading Procedures

Load tests were conducted in accordance with ASTM D-3689-83, Section 7.7, Quick Load Test Method (ASTM, 1991). Uplift loads were applied in increments of approximately 10 to 15% of the estimated shaft capacities with a constant time interval between load increments of 2-1/2 minutes. Load increments were added until continuous jacking was required or until the jack capacity was reached. Displacement readings were recorded immediately before and after each increment of load was added. Strain gauge readings and axial load were monitored continuously during each test.

INTERFACE DIRECT SHEAR TEST

Interface direct shear tests were conducted on samples from each site to investigate the applicability of the test for estimating design values of side resistance. The soil/cement mortar interface was used to model the interaction between a concrete shaft and the surrounding soil.

Tests were conducted using an ELE direct shear device at the University of Wyoming. A new shear box was fabricated to accomodate cylindrical samples obtained by the SDDOT plug sampler. Chuang and Reese (1969) found that the lowest shear strength for soil/cement mortar direct shear tests occurred 1/4 inch from the interface, in the soil. Therefore, the failure plane (boundary between the upper and lower halves of the box) was located 1/4 inch from the soil/cement interface. Figure 3-13 shows the direct shear box.

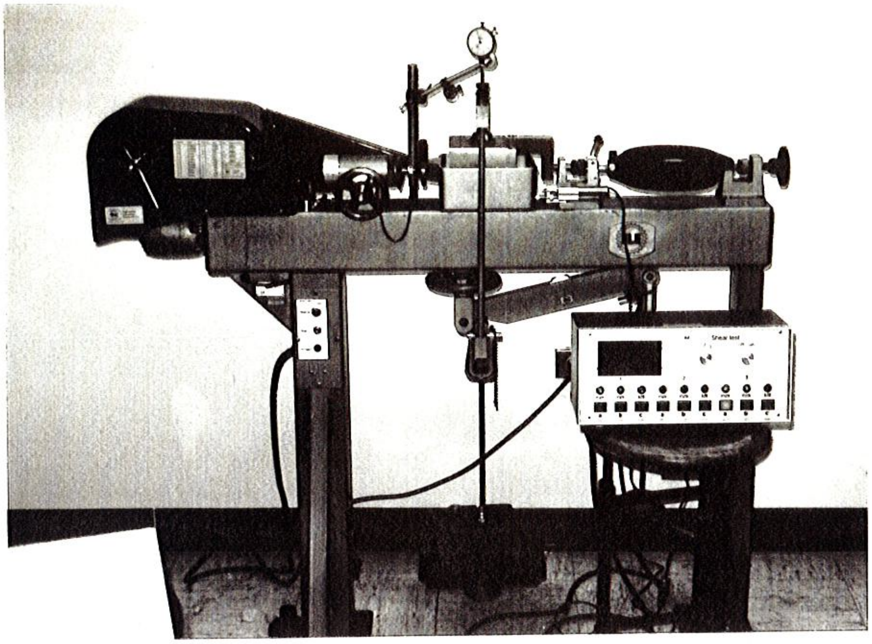


Figure 3-13. Interface Direct Shear Apparatus

The normal force applied to the test specimen was calculated by:

$$N_f = z\,\gamma_s\,A \tag{3-1}$$

where: N_f = normal force (lb), z = depth at which the sample was obtained (ft), γ_s = unit weight of soil (lb/ft^3), and A = cross sectional area of the specimen (ft^2).

Procedure

Each specimen was sliced, using a hacksaw blade, to the desired thickness and placed into the lower half of the shear box, which was then placed in the direct shear machine. Graphite powder, used to lubricate the surface between the upper and lower halves of the shear box, was applied to the lower shear box. The upper half of the shear box was then placed on top of the half containing the soil sample. Alignment pins were used to stabilize the shear box. Cement mortar was mixed and then poured directly onto the soil sample and allowed to cure for seven days. The normal force, N_f, was applied to the sample immediately after the cement mortar was poured to simulate the normal stress on the side of the shaft.

Following the curing period, the shearing portion of the test was conducted. Shearing load was applied at a displacement rate of 0.05 inches/minute, as recommended by Lambe (1951). Recordings of vertical and horizontal displacements and shearing force were taken every 10 seconds. The test was discontinued when the shearing force decreased from its peak value.

A minimum of two interface direct shear tests were conducted with specimens from each site. Specimens dimensions were approximately 2 inch diameter and 1-1/2 inch thick. The cement mortar had a water to cement ratio of 0.4 (the same W/C ratio used to construct the test shafts), and a sand to cement ratio of 1:3 based on recommendations of Chuang and Reese (1969).

Chapter IV

ANALYSIS OF RESULTS

Results of field uplift load tests on drilled shafts are presented. Side resistances from the load tests are compared to side resistances determined on the basis of the SDDOT pullout test, and to side resistances predicted by methods based on soil properties and from interface direct shear tests.

LOAD TEST RESULTS

Analysis of Uplift Capacities

Load versus displacement curves were plotted in order to determine the failure load of each test shaft, and these are presented in Figures 4-1 through 4-6. Failure loads for test shafts at sites A, D, and E were easily interpreted because each exhibits either a well-defined peak load or a horizontal asymptote corresponding to the maximum uplift load. Failure loads at Site B, C, and F do not exhibit a well-defined maximum load, and require interpretation.

Methods for interpreting failure loads from uplift tests on deep foundations have been reviewed by Hirany and Kulhawy (1988). Analysis of several methods using results of numerous uplift load tests on drilled shafts led to the conclusion that the load corresponding to a displacement of 0.5 in. is a reasonable approximation of the failure load. This method was used for the load tests at Sites B and C. At Site F, loading was terminated before reaching 0.5 in. of displacement. The failure loads for shafts 1 and 3 are

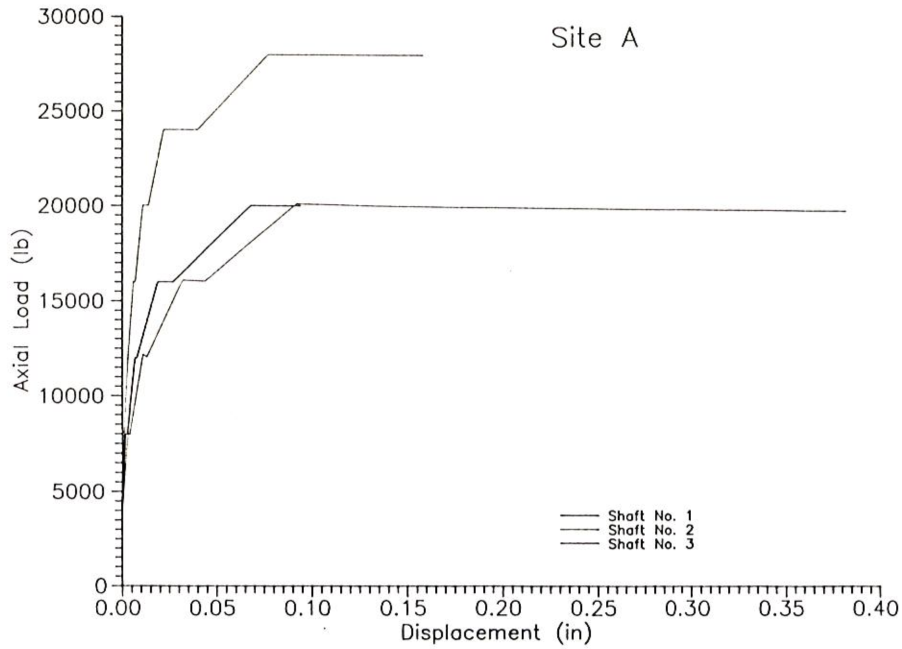


Figure 4-1. Load versus Displacement, Shafts at Site A.

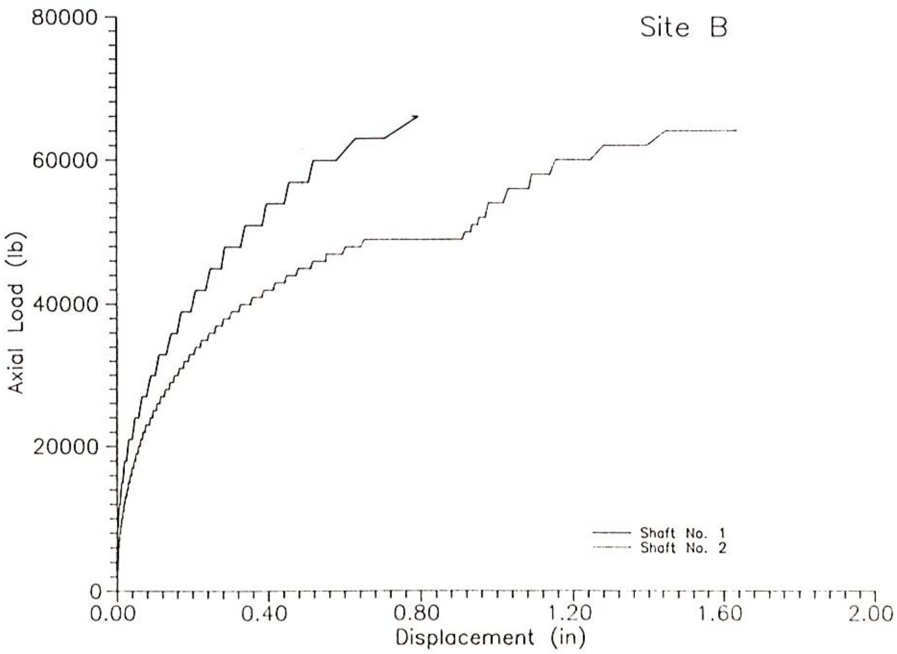


Figure 4-2. Load versus Displacement, Shafts at Site B.

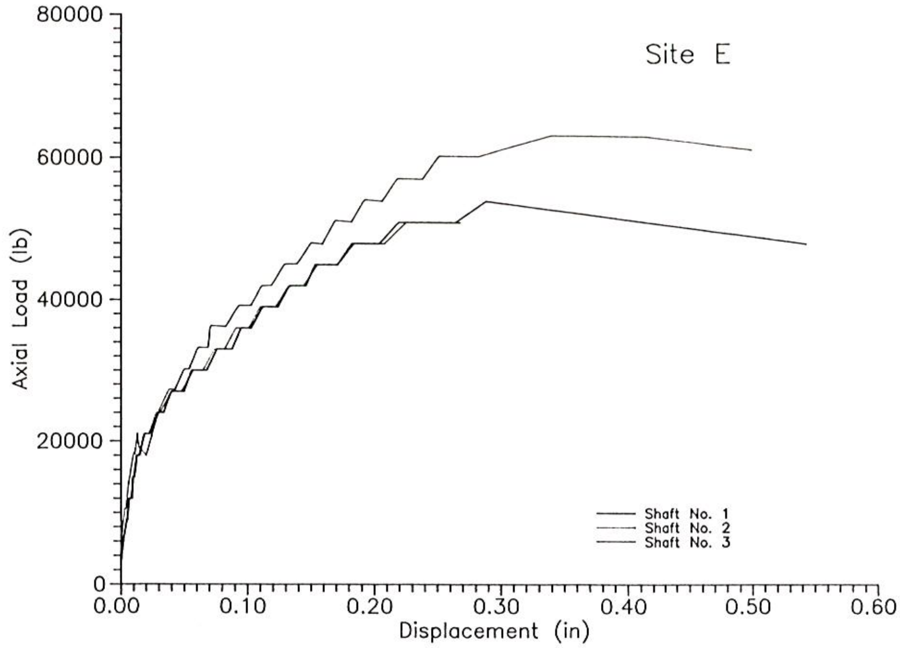


Figure 4-5. Load versus Displacement, Shafts at Site E.

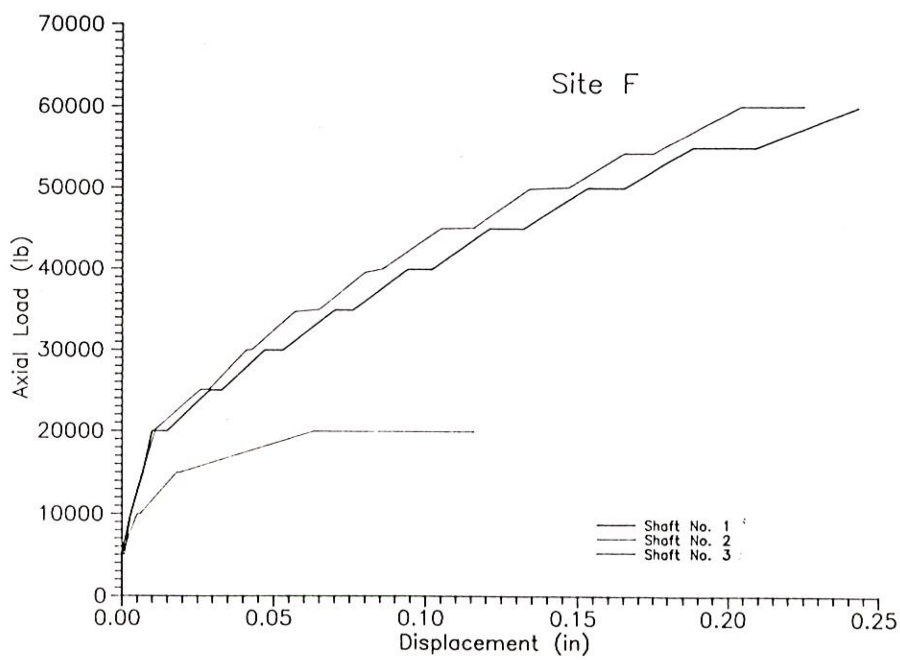


Figure 4-6. Load versus Displacement, Shafts at Site F.

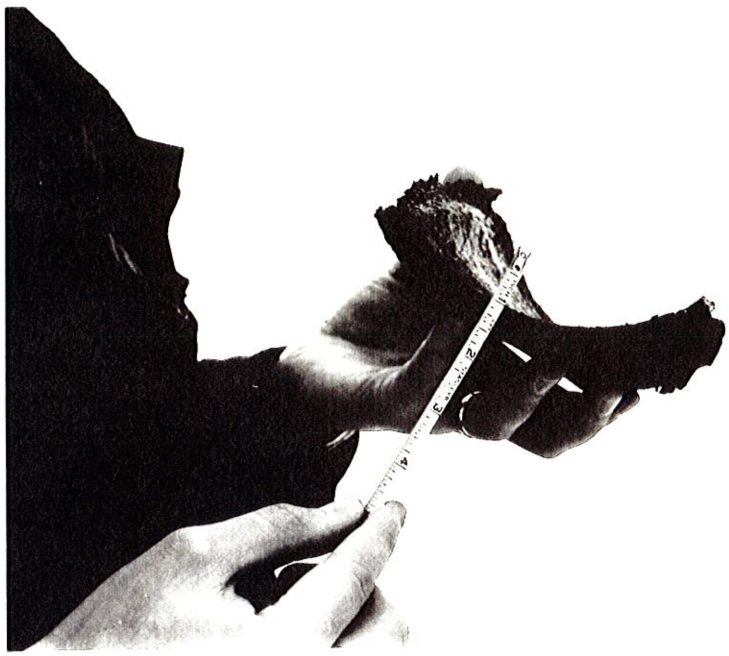


Figure 4-7. Measuring Thickness of Soil Collar Removed from Excavated Test Shaft.

Figure 4-8. Excavated Test Shafts with Voids, Site A.

Figure 4-9. Test Shaft A2 Following Excavation Showing 2.2. ft. Void at Tip.

TABLE 4.2. As-Built Dimensions of Test Shafts

Site	Shaft No.	Average Diameter[1] (ft)	Effective Concrete Length[2] (ft)	Observations
A	1	0.500[3]	5.5	6.7 ft void at tip
	2	0.500[3]	15.8	2.2 ft void at tip
	3	0.500[3]	8.6	5.4 ft void at tip
B	1	0.652	16.9	no voids
	2	0.679	14.7	2.2 ft of voids
	3	0.681	9.9	7 ft of voids
C	1	0.499	12.0	no voids
	2	0.498	12.0	no voids
	3	0.499	12.0	no voids
D	1	0.524	15.0	3 ft void at tip
	2	0.493	17.7	4 in. void at tip
	3	0.531	18.0	no voids
E	1	0.524	15.2	no voids
	2	0.493	14.9	no voids
	3	0.531	15.1	no voids
F	1	0.535	6.0	void from 6 to 18 ft
	2	0.509	17.9	no voids
	3	0.504	16.1	2 ft void at tip

1 - Avg. dia. determined from circumference measurement, not including soil adhered to shaft.
2 - Length of void has been subtracted.
3 - Assumed effective diameter, not measured.

$$Q_{ts} \approx A(u_i + 1 \text{ atm}) \tag{4-2}$$

where Q_{ts} = tip suction, A = cross-sectional area of the foundation tip, u_i = initial pore water stress at the tip, and 1 atmosphere = 2,116 psf. Estimates of tip suction for the shafts of this study using Equation 4-2 are given in Table 4-3. The best estimate of water table elevation at each site, observed either during the site investigation (Chapter III) or following removal of the test shafts, was used to compute u_i.

TABLE 4-4. Unit Side Resistances from Field Load Tests

Shaft	Total Side Resistance, Q_s (lb)	Measured Unit Side Resistance, f_m (psf)	Avg. Unit Side Resistance, SDDOT Pull Test, f_p (psf)
A1	19,824	2294	3396
A2	27,495	1108	
A3	21,525	1341	
B1	56,110	1556	339
B2	44,144	1408	
B3	50,435	2381	
C1	30,411	1617	2116
C2	37,386	1991	
C3	20,898	1111	
D1	28,798	1167	2560
D2	23,408	854	
D3	28,261	941	
E1	52,941	2115	1384
E2	49,990	2167	
E3	61,958	2460	
F1	19,780	1962	1787
F2	58,779	2053	
F3	59,423	2331	

load tests. The pull test results overpredict unit side resistances at Sites A, C, and D and underpredict unit side resistances at Sites B, E, and F.

Figure 4-11 presents the same data, except that variability of the pull test results is represented by error bars. The small horizontal bar in the middle represents the mean value of unit side resistance from the pull test and the two short horizontal bars on each end of the vertical bar represent the range of the standard error of the mean. The standard error of the mean measures the extent to which sample means vary, and is defined as:

$$\sigma_x = \sigma / n \qquad (4\text{-}3)$$

where σ_x = standard error of the mean, σ = standard deviation, and n = size of the sample. Variabilty in the pull test results ranges from moderate (Sites B, C, and D) to relatively large (Sites A, E, and F).

A best-fit linear regression curve through the origin and the data points shown in Figure 4-10 yields the statistics given in Table 4-5. The low correlation coefficient indicates a large amount of scatter and low degree of linearity in the relationship between predicted and measured side resistances. These statistics essentially indicate that there is no meaningful relationship between side resistance measured by the pull test and side resistance of the test shafts, when results from all of the sites are considered.

TABLE 4-5. Linear Regression, Side Resistances from SDDOT Pull Test

Data Points	Regression Equation (ksf)	Correlation Coefficient
18	$f_p = .970\ f_m$	0.378

The pull test results were next analyzed within the context of the α method for sites in cohesive soils and the β method for the cohesionless site. Applying the α method involves scaling the results of the pull test by the ratio of α for drilled shafts to α for driven pipe piles, since the drill stem can be modeled as driven pile. Note in Figures 2-3

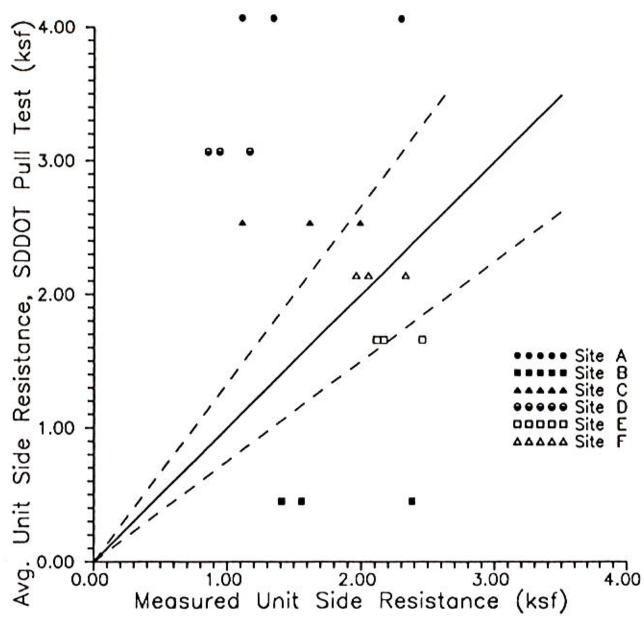


Figure 4-12. Predicted versus Measured Unit Side Resistances, Modified Pull Test Results.

side resistance or some other factors, such as construction problems, affected the test results.

SIDE RESISTANCE FROM SOIL PROPERTIES

An attempt was made to predict unit side resistance of the shafts in cohesive soils based upon measured undrained shear strengths, using the α method described in Chapter II. This method is limited by the fact that unconfined compression tests were conducted on samples obtained from the bottom few feet of the pull test holes only. A profile of undrained shear strength over the same depth as the test shafts is not available for any of the sites. Nevertheless, it is useful to compare this approach to the SDDOT pull test.

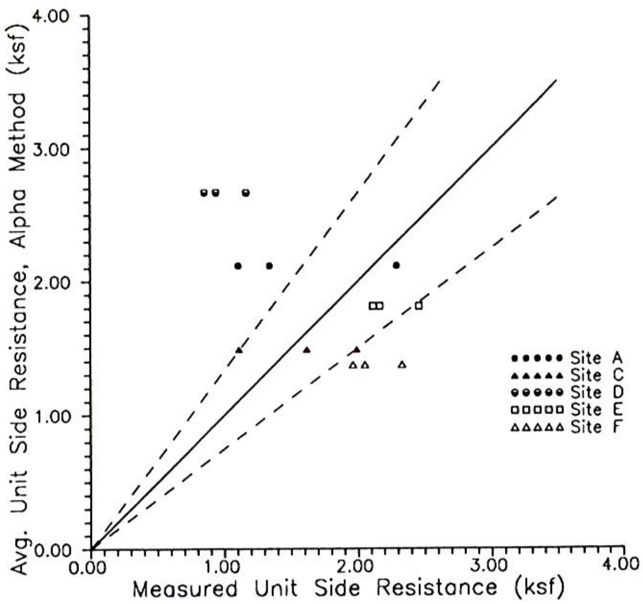


Figure 4-13. Predicted versus Measured Unit Side Resistances, α Method.

Results are given in Table 4-7. In this table, r is the coefficient of correlation, which is defined as the proportion of the total variation of the predicted values that is accounted for by the relationship with the measured values. A value of r close to 1 indicates a strong correlation between predicted and measured. Note that the two methods based on the pull test give the same value of $r = 0.71$, while the α method yields a low correlation coefficient. Based on these limited results, the pull test appears to yield the most reliable predictions, for sites without significant voids in the shafts.

INTERFACE DIRECT SHEAR TEST RESULTS

Interface direct shear tests were conducted to evaluate the possibility of developing a laboratory procedure for predicting side resistance. The apparatus and procedures are

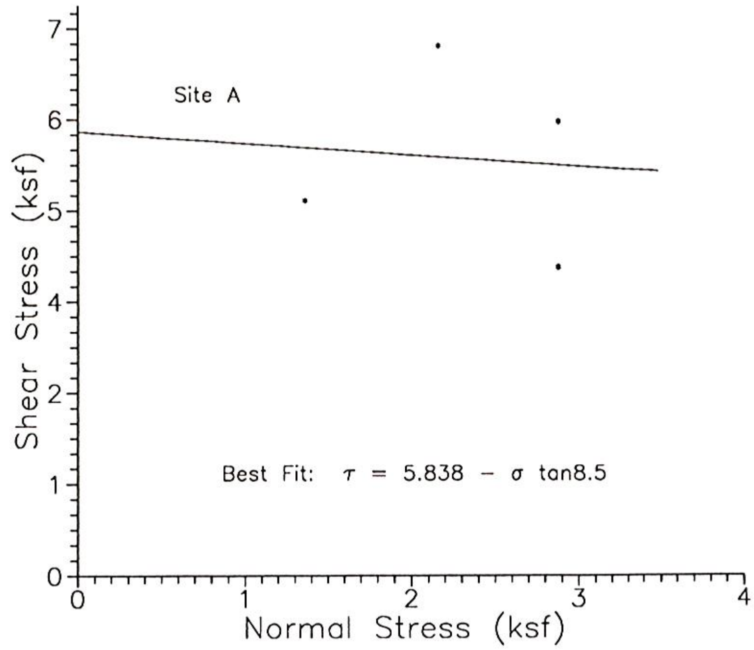


Figure 4-14. Interface Direct Shear Test Results, Site A

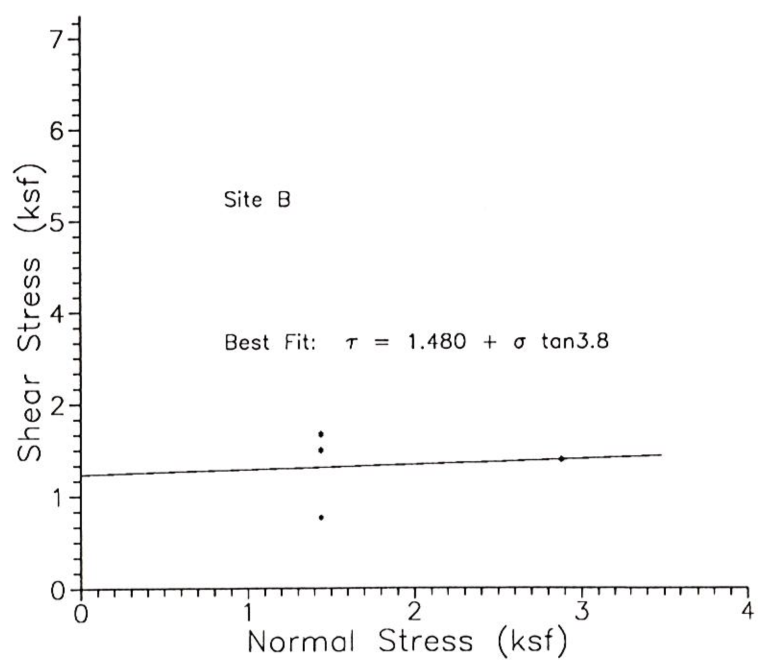


Figure 4-15. Interface Direct Shear Test Results, Site B

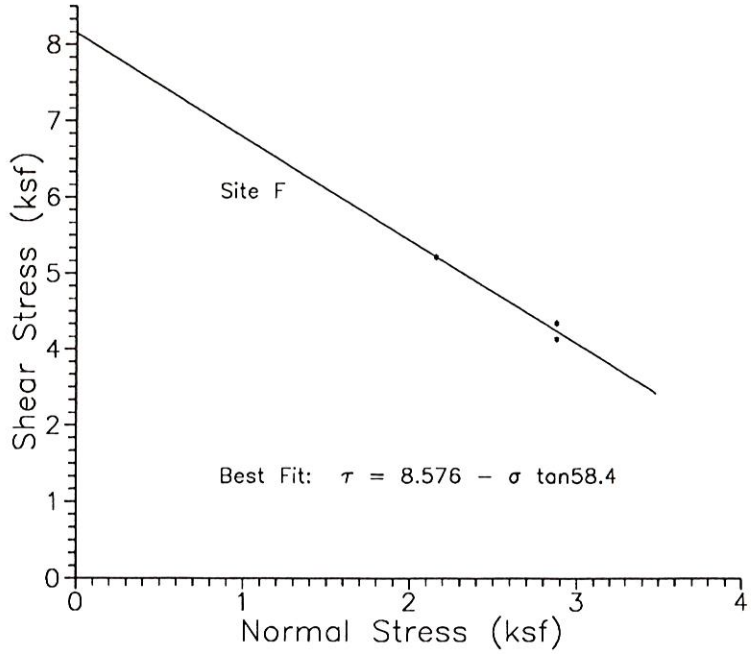


Figure 4-18. Interface Direct Shear Test Results, Site F

One factor that may lead to difficulties in using the interface direct shear test is the orientation of the specimens. A vertical deep foundation in compression or uplift results in a shearing surface that is approximately perpendicular to the bedding planes and fissures in the shales and weathered shales of this study. The cylindrical samples obtained using the retractable plug sampler of this study can only be tested in the direct shear apparatus with the failure plane parallel to the bedding planes and fissures, yielding lower shear strengths.

LOAD TRANSFER

Load transfer plots from the cohesive soil sites are shown in Figures 4-19 through 4-23. Load transfer plots are useful because they indicate the depths along which side resistance is being mobilized. At Site A, most of the load transfer occurred in the upper 5

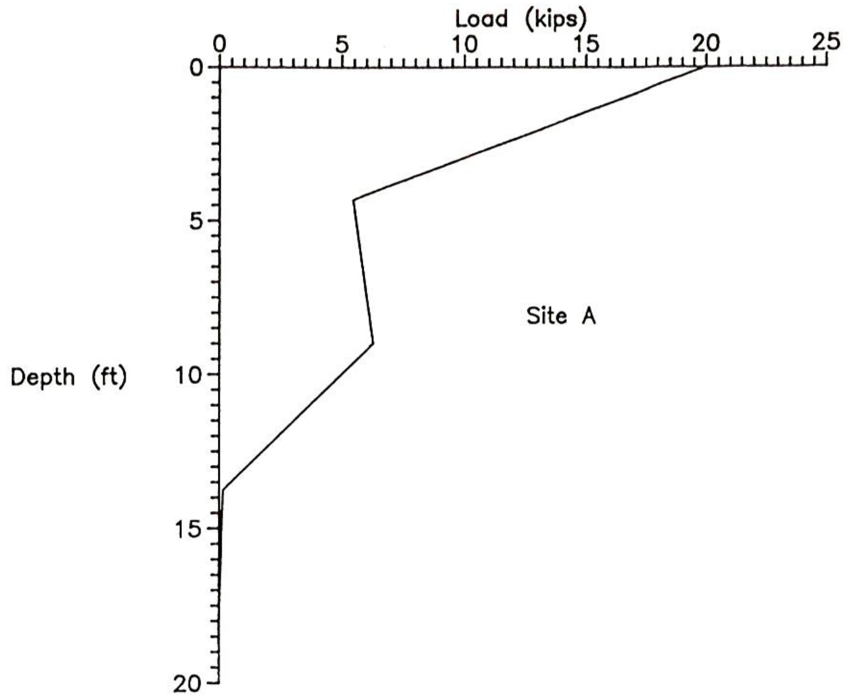


Figure 4-19. Load Transfer, Shaft A2.

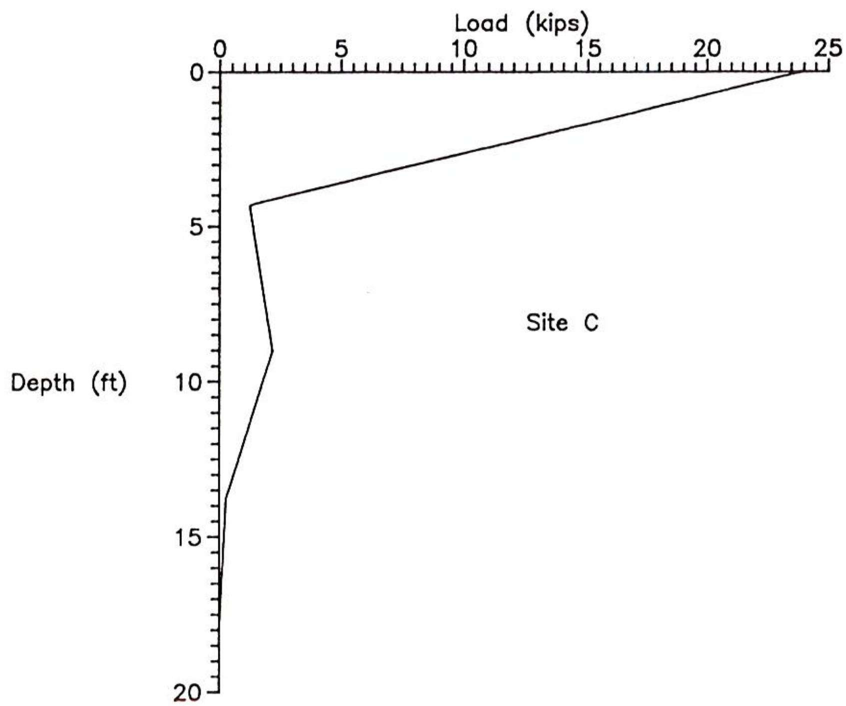


Figure 4-20. Load Transfer, Shaft C2.

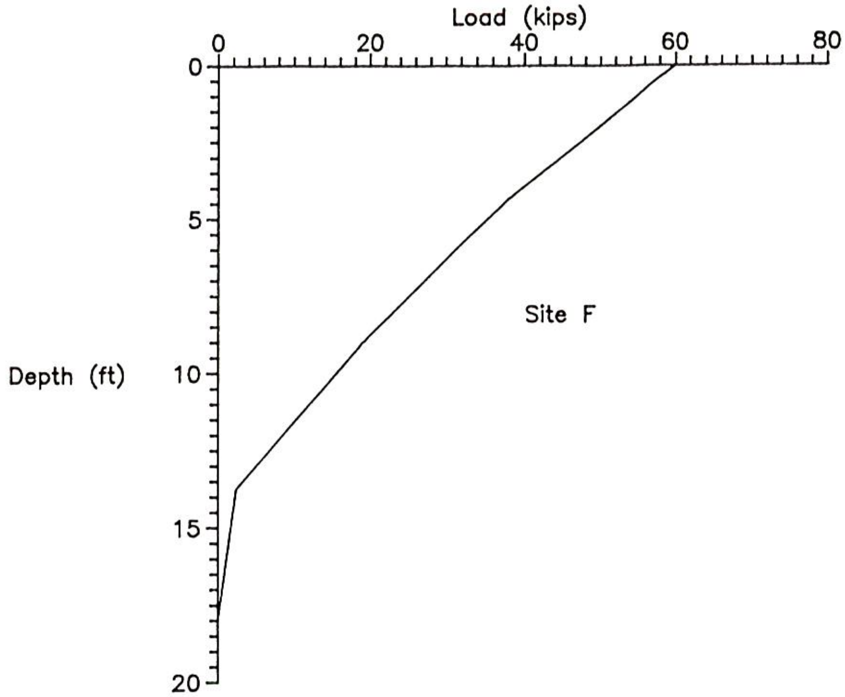


Figure 4-23. Load Transfer, Shaft F2.

Chapter V

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

SUMMARY

A study was conducted to achieve the following objectives:

1. Evaluate the current SDDOT field test method with respect to estimated versus actual side resistances for drilled shaft foundations.
2. Investigate the feasibility of using the direct shear laboratory test to estimate soil/shaft interface shearing resistance.
3. Develop recommendations and procedures for predicting side resistance, utilizing current SDDOT methods as much as possible.

A critical evaluation of the SDDOT field pull test was conducted, in which mechanics of the pull test are described within a rational framework that takes into account our current understanding of axial load transfer mechanisms for deep foundations. This analysis considers the interface material, foundation installation procedure, soil response to loading, and sources of variability. Example calculations are presented, using SDDOT test data, to illustrate how results of the pull test could be used to estimate the side resistance of drilled shafts for both cohesive and cohesionless soil sites.

Six field test sites were chosen in consultation with the SDDOT geotechnical staff. Five sites were selected in Rapid City, S.D., and included four cohesive and one agreement was obtained when unit side resistances predicted by the pull test were corrected for foundation type within the context of the α method.

Predictions of unit side resistance based on soil properties were conducted for the cohesive soil sites. Side resistances were computed based upon the mean undrained shear strength measured at the bottom of the pull test holes. The α method was employed, in which unit side resistance is taken as the product of the undrained shear strength and α obtained from Figure 2-4. These data also show considerable scatter and there is no consistent pattern of behavior. If only the sites with minor or no voids are considered (C, E, F), the scatter decreases and the measured side resistances are consistently higher (conservative) than the predicted, with most of the data falling close to or within 25 percent of predicted. This approach is limited by the fact that undrained shear strength values were obtained only along the lower 4 ft or less of each shaft. A complete profile of shear strength and its variation with depth might yield more meaningful correlations between shear strength and side resistance.

Load transfer diagrams for test shafts in the Belle Fourche, Spearfish, and Carlile weathered shales (Sites A, C, and D) indicate significant load transfer in the top 5 ft of the shafts, with little load transfer below that depth. This indicates much stiffer and stronger soil, and therefore more side resistance, in the near-surface zone, which may be due to the presence of overconsolidated crust. The water table at these sites was below five ft, with water content increasing with depth. An in-depth profile of soil shear strength with depth at these sites might yield useful information about side resistance and its variation with depth.

2. The presence of voids, such as were observed at Sites A and D of this study, generally results in much lower side resistances than predicted by the pull test. Problems with defective shafts decreased the reliability of the test results of this study.

3. Side resistance of drilled shafts determined by the α method, as recommended by Kulhawy (1992), yields moderately reliable, and conservative, correlations with side resistances measured in uplift load tests. As was the case for the pull test results, agreement between predicted and measured side resistances is good only for shafts at sites experiencing none or minor construction problems. The analysis conducted for this study is limited because soil undrained shear strengths were obtained only for a small interval along the foundation depths. This approach warrants further consideration.

4. The interface direct shear test does not appear to be a reliable procedure for obtaining deep foundation side resistances, for the materials tested in this study.

RECOMMENDATIONS

The pull test currently being used by SDDOT for prediction of side resistance offers several advantages and the authors recommend its continued use. The principal advantage of the pull test is that SDDOT personnel have been using it to design deep foundations since prior to 1964. Therefore, SDDOT has made a substantial investment in the pull test equipment, the operators are familiar with and experienced in the field procedures, and the engineering design professionals have been successfully incorporating the test results into their design recommendations for side resistance for many years. However, many advances

It is recommended that SDDOT consider other methods for predicting deep foundation side resistance, to be used in conjunction with the pull test. For sites in cohesive soils, the α method appears to offer promise, if reliable correlations can be developed for the weathered shales in South Dakota. Proper use of this method would require changes to the current SDDOT site characterization procedure. It would be necessary to obtain soil samples and conduct strength tests at locations other than the anticipated tip elevations. A profile of undrained shear strength, for example every 5 ft, is recommended.

Finally, it is recommended that SDDOT consider load testing of full-sized drilled shafts to evaluate design methods for determination of side resistance. Final verification of any method, whether it be the pull test or another method, is possible only under full-scale field construction and loading conditions. Based upon the load transfer diagrams obtained in this study, it appears that side resistance of the test shafts was influenced heavily by the crust of stiff soil in the upper 5 ft. This is a significant percentage of the length of the test shafts. Full-sized drilled shafts may be tens of feet in length and the influence of the top 5 ft may be minor. Construction of full-size test shafts should be conducted by an experienced and qualified drilled shaft contractor in order to simulate production conditions and to obtain shafts without voids. In conjunction with the additional small-scale load tests recommended above, several full-scale load tests can provide a reliable data base of field performance upon which to evaluate the pull test, as well as other methods for predicting side resistance.

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APPENDIX A

LOAD TEST DATA

This appendix contains copies of completed SDDOT Form C, Load Test Data Sheet.

FORM C
LOAD TEST DATA SHEET

Project		Test Shaft No	A-1	Sheet 1 of 1	
Location	SDSM&T	Shaft Dia.		Temp.	30's
Date	12/4/91	Tip elev.		Weather	cldy.
Engr/Tech		Butt elev.		Hyd. Jack No.	
Remarks		Ground elev.		Pres. gage. No.	

Zero Gauge - No Load

Clock Time	Elaps. Time (min.)	Reqd. Load (lb.)	Hyd. Jack (tons)	Gage 1 reading (in.)	Gage 2 reading (in.)	mean	Remarks
	0.0	4000	0	.998	.998		
	2.5	4000	0	.998	.998		
	0.0	8000	0	.996	.996		
	2.5	8000	0	.995	.995		
	0.0	12,000	2.9TN	.991	.991		
	2.5	12,000	2.9TN	.990	.990		
	0.0	16,000	4.3TN	.979	.979		
	2.5	16,000	4.5TN	.971	.971		
	0.0	20,000	6.1TN	.930	.930		
	2.5	20,000	6.1TN	.905	.903		
			FAILED				
		18,300		.722	.720		
		0		.753	.755		

FORM C
LOAD TEST DATA SHEET

Project		Test Shaft No.	A-2	Sheet	1 of 1
Location	SDSM&T	Shaft Dia.		Temp.	30's
Date	12/5/91	Tip elev.		Weather	CLEAR
Engr/Tech		Butt elev.		Hyd. Jack No.	
Remarks		Ground elev.		Pres. gage. No.	

No Load Gauge Zeroed

Clock Time	Elaps. Time (min)	Reqd. Load (lb.)	Hyd. Jack (tons)	Gage 1 reading (in.)	Gage 2 reading (in.)	mean	Remarks scan
	0.0	4000	0	.999	.999		
	2.5	4000	0	.999	.999		
	0.0	8000	1.9	.998	.998		
	2.5	8000	1.9	.998	.998		
	0.0	12,000	3.1	.996	.996		
	2.5	12,000	3.1	.996	.996		
	0.0	16,000	5.0	.993	.993		
	2.5	16,000	5.0	.992	.992		
	0.0	20,000	6.6	.988	.988		
	2.5	20,000	6.8	.985	.985		
	0.0	24,000	8.1	.977	.977		
	2.5	24,000	8.1	.959	.959		
	0.0	28,000	10.0	.922	.920		
	2.5	28,000	10.0	.841	.839		
			FAILED				
		28,500		.541	.539		
		0		.567	.566		

FORM C
LOAD TEST DATA SHEET

Project SD 9015	Test Shaft No. B-1(East)	Sheet 2 of 3
Location 5th St	Shaft Dia.	Temp. 40's
Date 12/12/91	Tip elev.	Weather
Engr/Tech	Butt elev.	Hyd. Jack No.
Remarks	Ground elev.	Pres. gage. No.

Clock Time	Elaps. Time (min.)	Reqd. Load (lb.)	Hyd. Jack (tons)	Gage 1 reading (in.)	Gage 2 reading (in.)	mean	Remarks
	0.0	33000	12.5	.885	.890		
	2.5	33000	12.5	.866	.872		
	0.0	36000	14.0	.853	.858		
	2.5	36000	14.0	.836	.842		
	0.0	39000	15.2	.826	.831		
	2.5	39000	15.2	.799	.805		
	0.0	42000	17.0	.787	.794		
	2.5	42000	17.0	.760	.767		
	0.0	45000	18.2	.748	.754		
	2.5	45000	18.2	.719	.726		
	0.0	48000	19.9	.708	.718		
	2.5	48000	19.9	.667	.676		
	0.0	51000	21.2	.655	.664		
	2.5	51000	21.2	.609	.619		
	0.0	54000	27.7	.599	.608		
	2.5	54000	27.7	.551	.561		
	0.0	57000	24.0	.539	.549		
	2.5	57000	24.0	.487	.499		
	0.0	60000	25.6	.475	.485		

FORM C
LOAD TEST DATA SHEET

Project		Test Shaft No.	B-2	Sheet 1 of 7	
Location	5th St. - Site B	Shaft Dia.		Temp.	20's
Date	12/12/91	Tip elev.		Weather	CLEAR
Engr/Tech		Butt elev.		Hyd. Jack No.	
Remarks		Ground elev.		Pres. gage. No.	

Zero Gauge - No Load

Clock Time	Elaps. Time (min)	Reqd. Load (lb.)	Hyd. Jack (tons)	Gage 1 reading (in.)	Gage 2 reading (in.)	mean	Remarks
	0.0	1000	-	.9995	.9995		
	2.5	1000	-	.9995	.9995		
	0.0	2000	-	.999	.999		
	2.5	2000	-	.9985	.999		
	0.0	3000	-	.998	.998		
	2.5	3000	-	.998	.9975		
	0.0	4000	-	.997	.997		
	2.5	4000	-	.997	.997		
	0.0	5000	1TN	.9965	.996		
	2.5	5000	1TN	.996	.9955		
	0.0	6000	1.2TN	.995	.995		
	2.5	6000	1.2TN	.9945	.994		
	0.0	7000	1.9TN	.994	.993		
	2.5	7000	1.9	.993	.9925		
	0.0	8000	2.1	.992	.991		
	2.5	8000	2.1	.991	.990		
	0.0	9000	2.8	.990	.989		
	2.5	9000	2.8	.9885	.988		
	0.0	10000	3.0	.9875	.986		
	2.5	10000	3.0	.986	.9845		

FORM C
LOAD TEST DATA SHEET

Project		Test Shaft No.	B-2	Sheet 3 of 7	
Location	5th St.	Shaft Dia.		Temp.	20's
Date	12/12/91	Tip elev.		Weather	CLEAR
Engr/Tech		Butt elev.		Hyd. Jack No.	
Remarks		Ground elev.		Pres. gage. No.	

Zero Gauge - No Load

Clock Time	Elaps. Time (min.)	Reqd. Load (lb.)	Hyd. Jack (tons)	Gage 1 reading (in.)	Gage 2 reading (in.)	mean	Remarks
	2.5	20000	7.6	.9415	.936		
	0.0	21000	8.0	.939	.934		
	2.5	21000	8.1	.935	.929		
	0.0	22000	8.8	.933	.9265		
	2.5	22000	8.8	.928	.921		
	0.0	23000	9.1	.926	.919		
	2.5	23000	9.1	.919	.907		
	0.0	24000	9.8	.917	.905		
	2.5	24000	9.8	.911	.898		
	0.0	25000	10	.909	.8955		
	2.5	25000	10	.902	.889		
	0.0	26000	10.5	.900	.886		
	2.5	26000	10.6	.8925	.879		
	0.0	27000	11.1	.891	.876		
	2.5	27000	11.1	.882	.866		
	0.0	28000	11.8	.879	.863		
	2.5	28000	11.8	.870	.853		
	0.0	29000	12.1	.868	.851		
	2.5	29000		.859	.841		

FORM C
LOAD TEST DATA SHEET

Project		Test Shaft No.	B-2	Sheet 5 of 7	
Location	5th St.	Shaft Dia.		Temp.	20's
Date	12/12/91	Tip elev.		Weather	CLEAR
Engr/Tech		Butt elev.		Hyd. Jack No.	
Remarks		Ground elev.		Pres. gage. No.	

Zero Gauge - No Load

Clock Time	Elaps. Time (min.)	Reqd. Load (lb.)	Hyd. Jack (tons)	Gage 1 reading (in.)	Gage 2 reading (in.)	mean	Remarks
	2.5	39000	17.8	.691	.662		
	0.0	40000	18.1	.687	.658		
	2.5	40000	18.1	.660	.630		
	0.0	41000	18.5	.657	.626		
	2.5	41000	18.5	.630	.600		
	0.0	42000	19.1	.627	.595		
	2.5	42000	19.1	.602	.569		
	0.0	43000	19.8	.598	.565		
	2.5	43000	19.8	.572	.539		
	0.0	44000	20	.568	.535		
	2.5	44000	20	.542	.508		
	0.0	45000	20.5	.538	.503		
	2.5	45000	20.5	.505	.470		
	0.0	46000	21.0	.500	.464		
	2.5	46000	21.0	.465	.430		
	0.0	47000	21.8	.460	.424		
	2.5	47000	21.8	.424	.384		
	0.0	48000	22.0	.416	.379		
	2.5	48000	22.1	.375	.338		

FORM C
LOAD TEST DATA SHEET

Project ____________
Location 5th ST.
Date 12/12/91
Engr/Tech ____________
Remarks ____________

Test Shaft No. B-2
Shaft Dia. ____________
Tip elev. ____________
Butt elev. ____________
Ground elev. ____________

Sheet 7 of 7
Temp. 20's
Weather CLEAR
Hyd. Jack No. ____
Pres. gage. No. ____

Zero Gauge - No Load

Clock Time	Elaps. Time (min.)	Reqd. Load (lb.)	Hyd. Jack (tons)	Gage 1 reading (in.)	Gage 2 reading (in.)	mean	Remarks
	2.5	58000	24.5	.944	.943		
	0.0	60000	25.6	.929	.928		
	2.5	60000	25.6	.839	.835		
	0.0	62000	26.8	.805	.800		
	2.5	62000	26.8	.691	.684		
	0.0	64000	27.5	.645	.625		
	2.5	64000	28.0	.460	.435		
			Assume	Failure			
		0		.696	.678		

FORM C
LOAD TEST DATA SHEET

Project SD 9015
Location 5th St. Rapid City
Date 12/12/91
Engr/Tech
Remarks

Test Shaft No. B-3
Shaft Dia.
Tip elev.
Butt elev.
Ground elev.

Sheet 2 of 3
Temp. 40's
Weather
Hyd. Jack No.
Pres. gage. No.

Zero Gauge - No Load

Clock Time	Elaps. Time (min.)	Reqd. Load (lbs)	Hyd. Jack (tons)	Gage 1 reading (in.)	Gage 2 reading (in.)	mean	Remarks
	2.5	30000	10.2	.902	.888		
	0.0	33000	11.7	.892	.875		
	2.5	33000	11.7	.872	.847		
	0.0	36000	13.0	.864	.836		
	2.5	36000	13.0	.839	.810		
	0.0	39000	14.2	.830	.798		
	2.5	39000	14.2	.794	.761		
	0.0	42000	15.8	.785	.751		
	2.5	42000	15.8	.753	.718		
	0.0	45000	17.2	.740	.702		
	2.5	45000	17.2	.675	.638		
	0.0	48000	18.8	.660	.620		
	2.5	48000	18.8	.596	.556		
	0.0	51000	20.2	.580	.538		
	2.5	51000	20.2	.501	.458		
	0.0	54000	21.8	.477	.432		
	2.5	54000	21.8	.406	.361		
	0.0	57000	23.0	.387	.341		
	2.5	57000	23.0	.205	.157		

FORM C
LOAD TEST DATA SHEET

Project		Test Shaft No.	C-1	Sheet 1 of 2	
Location	Deadwood Ave.	Shaft Dia.		Temp.	30
Date	12/4/91	Tip elev.		Weather	PC
Engr/Tech		Butt elev.		Hyd. Jack No.	
Remarks		Ground elev.		Pres. gage. No.	

SD9015 Dϕ7

Clock Time	Elaps. Time (min.)	Reqd. Load (lb.)	Hyd. Jack (tons)	Gage 1 reading (in.)	Gage 2 reading (in.)	mean	Remarks Scan #
	0.0	4000	0	.998	.998		12
	2.5	4000	0	.998	.998		90
	0.0	8000	2TN	.993	.992		97
	2.5	8000	2TN	.990	.989		172
	0.0	12000	3.9TN	.970	.969		179
	2.5	12000	3.9TN	.950	.949		255
	0.0	16000	5.1TN	.926	.925		264
	2.5	16000	5.1TN	.895	.893		337
	0.0	20000	7.1TN	.852	.850		348
	2.5	20000	7.0TN	.806	.800		425
	0.0	24000	9.0TN	.760	.752		436
	2.5	24000	9.0TN	.696	.689		512
	0.0	28000	10.9TN	.630	.624		529
	2.5	28000	10.5TN	.560	.554		604
	0.0	32000	12.5TN	.500	.490		618
	2.5	32000	12.6TN	.412	.404		692
	0.0	36000	14.2TN	.333	.324		706
	2.5	36000	14.2TN	.206	.197		782

FORM C
LOAD TEST DATA SHEET

Project		Test Shaft No. C-2	Sheet 1 of 2
Location	Deadwood Ave.	Shaft Dia.	Temp. 30's
Date	12/4/91	Tip elev.	Weather PC
Engr/Tech		Butt elev.	Hyd. Jack No.
Remarks		Ground elev.	Pres. gage. No.

Zero Gauge - No Load

Clock Time	Elaps. Time (min.)	Reqd. Load (lb.)	Hyd. Jack (tons)	Gage 1 reading (in.)	Gage 2 reading (in.)	mean	Remarks
	0.0	4000	0	.999	.998		
	2.5	4000	0	.998	.998		
	0.0	8000	1TN	.996	.996		
	2.5	8000	1TN	.993	.993		
	0.0	12000	3TN	.980	.980		
	2.5	12000	3TN	.970	.970		
	0.0	16000	4.6TN	.955	.954		
	2.5	16000	4.6TN	.940	.939		
	0.0	20000	6.2TN	.920	.920		
	2.5	20000	6.2TN	.894	.894		
	0.0	24000	8.0TN	.877	.875		
	2.5	24000	8.0TN	.834	.834		
	0.0	28000	10. TN	.800	.800		
	2.5	28000	10. TN	.772	.773		
	0.0	32000	11.9TN	.733	.730		Moveme nt Increase
	2.5	32000	11.9TN	.655	.654		
	0.0	36000	13.4TN	.605	.605		
	2.5	36000	13.5TN	.543	.543		
	0.0	40000	15.1TN	.485	.485		

FORM C
LOAD TEST DATA SHEET

Project ______
Location DEADWOOD AVE.
Date 12-4-91
Engr/Tech ______
Remarks ______

Test Shaft No. C-3
Shaft Dia. ______
Tip elev. ______
Butt elev. ______
Ground elev. ______

Sheet 1 of 2
Temp. TEENS to TWENTY
Weather CLEAR
Hyd. Jack No. ____
Pres. gage. No. ____

Zero Gauge - No Load

Clock Time	Elaps. Time (min.)	Reqd. Load (lb.)	Hyd. Jack (tons)	Gage 1 reading (in.)	Gage 2 reading (in.)	mean	Remarks
	0.0	4000	0	.998	.998		
	2.5	4000	0	.998	.998		
	0.0	8000	2.6TN	.995	.995		
	2.5	8000	2.8TN	.994	.993		
	0.0	12000	4.1TN	.985	.984		
	2.5	12000	4.1TN	.969	.968		
	0.0	16000	6.0TN	.938	.936		Pump to Hold
	2.5	16000	6.0TN	.884	.885		
	0.0	20000	7.9TN	.820	.818		
	2.5	20000	7.9TN	.723	.722		
	0.0	24000	9.9TN	.617	.615		
	2.5	24000	9.9TN	.473	.467		
	0.0	28000	11.9TN	.378	.373		Before Move Up
	2.5	28000	11.9TN	.181	.176	.162	.155
	0.0	32000	13.8TN	.952	.945		
			Rezeroed	Before	Using		
	2.5	32000	13.9TN	.758	.757		
	0.0	36000	15.8TN	.680	.677		
	2.5	36000	15.9TN	.468	.468		

FORM C
LOAD TEST DATA SHEET

Project		Test Shaft No. D-1 (WEST)	Sheet 1 of 1
Location	La Crosse	Shaft Dia.	Temp.
Date	12/3/91	Tip elev.	Weather
Engr/Tech		Butt elev.	Hyd. Jack No.
Remarks		Ground elev.	Pres. gage. No.

Zero Gauge - No Load

Clock Time	Elaps. Time (min.)	Reqd. Load (lb.)	Hyd. Jack (tons)	Gage 1 reading (in.)	Gage 2 reading (in.)	mean	Remarks
	0	4000		.9980	.9985		
	2.5 ?	4000		.997	.997		
	0	8000	1.0	.994	.995		
	2.5	8000		.994	.995		
	0	12000	3.5	.990	.991		
	2.5	12000		.989	.990		
	0	16000	5.0	.984	.985		
	2.5	16000		.982	.983		
	0	20000	7.0	.975	.976		
	2.5	20000		.969	.971		
	0	24000	8.5	.951	.954		
	2.5	24000		.936	.936		
	0	28000	10.1	.910	.912		
	2.5	28000		.839	.846		
2900	0-30 sec	29250	11.0	.620	.625		
	2.5	24012 12 tons		.593	.598		
		0	0	.651	.651		

FORM C
LOAD TEST DATA SHEET

Project ________
Location La Crosse
Date 12/3/91
Engr/Tech ________
Remarks ________

Test Shaft No. D-3 (EAST)
Shaft Dia. ________
Tip elev. ________
Butt elev. ________
Ground elev. ________

Sheet 1 of 1
Temp. TEENS
Weather PC - Snowy
Hyd. Jack No. ____
Pres. gage. No. ____

Dϕ6 File SD9015,Dϕ5

Clock Time	Elaps. Time (min.)	Reqd. Load (lb.)	Hyd. Jack (tons)	Gage 1 reading (in.)	Gage 2 reading (in.)	mean	Remarks Scan #
	0.0	5000	0	.999	.999		5
	2.5	5000	0	.999	.999		83
	0.0	10000	2.8TN	.995	.996		91
	2.5	10000	2.8TN	.995	.995		170
	0.0	15000	4.8TN	.991	.991		191 ?
	2.5	15000	4.8TN	.989	.990		251
	0.0	20000	7.0TN	.983	.983		264
	2.5	20000	7.0TN	.977	.977		335
	0.0	25000	9.0TN	.964	.964		345
	2.5	25000	9.0TN	.944	.945		421
	0.0	30000	11.1TN	.914	.914		435
	2.5	30000	11.1TN	.744	.744		505
		REBOUND	FAILURE				
		28250		.501	.500		561
		0		.551	.551		

FORM C
LOAD TEST DATA SHEET

Project		Test Shaft No.	E-1	Sheet	2 of 2
Location	St. Pat.	Shaft Dia.		Temp.	TEENS
Date	12/3/91	Tip elev.		Weather	
Engr/Tech		Butt elev.		Hyd. Jack No.	
Remarks		Ground elev.		Pres. gage. No.	

Zero Gauge - No Load

Clock Time	Elaps. Time (min.)	Reqd. Load (lb.)	Hyd. Jack (tons)	Gage 1 reading (in.)	Gage 2 reading (in.)	mean	Remarks
	2.5	30000		.931	.927		
	0.0	33000	13.0	.924	.919		
	2.5	33000		.912	.906		
	0.0	36000	14.2	.905	.899		
	2.5	36000		.898	.890		
	0.0	39000	15.5	.889	.881		
	2.5	39000		.878	.869		
	0.0	42000	17.0	.869	.860		
	2.5	42000		.856	.847		
	0.0	45000	18.3	.849	.840		
	2.5	45000		.831	.821		
	0.0	48000	20.0	.821	.810		
	2.5	48000		.799	.788		
	0.0	51000	21.2	.784	.772		
	2.5	51000		.739	.728		
	0.0	54000	22.5	.716	.703		
		48000		.458	.451		
	0.0	0	0	.592	.586		

FORM C
LOAD TEST DATA SHEET

Project ________
Location St. Pat.
Date 12/2/91
Engr/Tech ________
Remarks ________

Test Shaft No. E-2
Shaft Dia. ________
Tip elev. ________
Butt elev. ________
Ground elev. ________

Sheet 2 of 2
Temp. TEENS
Weather V. Windy, Cldy.
Hyd. Jack No. ____
Pres. gage. No. ____

Zero Gauge - No Load

Clock Time	Elaps. Time (min.)	Reqd. Load (lb.)	Hyd. Jack (tons)	Gage 1 reading (in.)	Gage 2 reading (in.)	mean	Remarks
	2.5	30000		.933	.933		
	0.0	33000	12.1TN	.924	.924		
	2.5	33000		.916	.916		
	0.0	36000	13.6TN	.908	.907		
	2.5	36000		.898	.898		
	0.0	39000	15TN	.889	.887		
	2.5	39000		.878	.875		
	0.0	42000	17TN	.869	.865		
	2.5	42000		.857	.852		
	0.0	45000	18TN	.846	.841		
	2.5	45000		.830	.824		
	0.0	48000	19TN	.818	.810		
	2.5	48000		.794	.786		
	0.0	51000	21TN	.779	.768		
	2.5	51000		.737	.723		
		rebound					
				.622	.605		
				.765	.759		

Failed between 51000 and 54000 - Could nearly hold 54000.

FORM C
LOAD TEST DATA SHEET

Project ________ Test Shaft No. E-3 Sheet 2 of 3
Location St. Pat. Shaft Dia. ________ Temp. 10°
Date 12/3/91 Tip elev. ________ Weather Snow & Breezy
Engr/Tech ________ Butt elev. ________ Hyd. Jack No. ____
Remarks ________ Ground elev. ________ Pres. gage. No. ____

Zero Gauge - No Load Data file SD9015.Dϕ4

Clock Time	Elaps. Time (min.)	Reqd. Load (lb.)	Hyd. Jack (tons)	Gage 1 reading (in.)	Gage 2 reading (in.)	mean	Remarks Scan #
	2.5	30000		.942	.946		828
	0.0	33000	12.2	.935	.940		835
	2.5	33000	12.2	.926	.932		911
	0.0	36000		.919	.924		917
	2.5	36000		.912	.918		993
	0.0	39000	15.1	.901	.908		1001
	2.5	39000	15.1	.891	.899		1077
	0.0	42000	16.7	.883	.891		1084
	2.5	42000	16.8	.875	.884		1159
	0.0	45000	18.1	.865	.874		1166
	2.5	45000	18.1	.855	.864		1243
	0.0	48000	19.8	.844	.853		1251
	2.5	48000	19.8	.834	.845		1327
	0.0	51000	21.1	.824	.835		1335
	2.5	51000	21.1	.811	.822		1411
	0.0	54000	22.5	.801	.812		1419
	2.5	54000	22.5	.786	.798		1495
	0.0	57000	24.0	.774	.786		1501 Pump to Hold
	2.5	57000	24.0	.754	.766		1577

FORM C
LOAD TEST DATA SHEET

Project
Location PIERRE
Date 11/26/91
Engr/Tech Longhons
Remarks

Test Shaft No. F-1
Shaft Dia.
Tip elev.
Butt elev.
Ground elev.

Sheet 1 of 1
Temp. 40's
Weather WINDY
Hyd. Jack No.
Pres. gage. No.

Zero Gauge - No Load

Clock Time	Elaps. Time (min.)	Reqd. Load (lb.)	Hyd. Jack (tons)	Gage 1 reading (in.)	Gage 2 reading (in.)	mean	Remarks
	0.0	5000		.997	.997		
	2.5min	5000		.997	.997		
	0.0m	10000	2.5TN	.992	.991		
	2.5	10000	2.5TN	.991	.991		
	0.0	15000	5TN	.980	.979		
	2.5	15000	5TN	.978	.978		
	0.0	20000	7TN	.936	.932		
	2.5	20000	7TN	.882	.880		Pump to Hold
			FAILURE	at	22000	MAX	

Load cell and Jack Gauge had 2 to 2.6 TN discrepancy could not load above 22,000#.

Gauges zeroed with no load.

FORM C
LOAD TEST DATA SHEET

Project ____________ Test Shaft No. F-2 Sheet 2 of 2
Location PIERRE Shaft Dia. ____________ Temp. 40's
Date 11/26/91 Tip elev. ____________ Weather WINDY
Engr/Tech Longbons Butt elev. ____________ Hyd. Jack No. ____
Remarks ____________ Ground elev. ____________ Pres. gage. No. ____

____

Zero Gauge - No Load

Clock Time	Elaps. Time (min.)	Reqd. Load (lb.)	Hyd. Jack (tons)	Gage 1 reading (in.)	Gage 2 reading (in.)	mean	Remarks
	2.5	50000		.855	.849		910
	0.0	55000	22TN	.838	.830		935
	2.5	55000	22TN	.828	.820		1010
	0.0	60000	24.5TN	.800	.790		1027
	2.5	60000	24.75	.780	.769		1100
			Readjust because of Settling				
	0.0	65000	Failure				
			Rebound				
		61000	27TN	.455	.392		1384
		0.00		.684	.631		

Max Load Achieved

63,000

FORM C
LOAD TEST DATA SHEET

Project ________
Location PIERRE
Date 11/27/91
Engr/Tech Longbons
Remarks ________

Test Shaft No. F-3
Shaft Dia. ________
Tip elev. ________
Butt elev. ________
Ground elev. ________

Sheet 2 of 2
Temp. High 20's - Low 30's
Weather SUNNY
Hyd. Jack No. ____
Pres. gage. No. ____

Zero Gauge - No Load

Clock Time	Elaps. Time (min.)	Reqd. Load (lb.)	Hyd. Jack (tons)	Gage 1 reading (in.)	Gage 2 reading (in.)	mean	Remarks
	2.5	50000		.834	.830		
	0.0	55000	24TN	.811	.808		
	2.5	55000		.790	.786		
	0.00	60000	26.5TN	.754	.754		
	2.5	60000	Failure				
		60000 MAX					
			Rebound				
				.445	.445		
		0		.585	.581		